1-1-1972

Injection of sediment containing water into permeable unconfined formations.

Safa Aldin Jubboori
University of Massachusetts Amherst

INJECTION OF SEDIMENT CONTAINING WATER INTO PERMEABLE UNCONFINED FORMATIONS

A Dissertation Presented

By

SAFA ALDIN JUBBOORI

Submitted to the Graduate School of the
University of Massachusetts in
partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

February 1972

Major Subject: Soil Physics

INJECTION OF SEDIMENT CONTAINING WATER INTO

PERMEABLE UNCONFINED FORMATIONS

A Dissertation

By

SAFA ALDIN JUBBOORI

Approved as to style and content by:

Gordon L. Stewart
(Chairman of Committee)

F.W. Southwick
(Head of Department)

Donald Dean Adrian
(Member)

J. H. Baker
(Member)

February 1972

ACKNOWLEDGMENTS

I am indebted to both Dr. G. L. Stewart and Dr. D. D. Adrian for their valuable guidance throughout this investigation. Also I wish to express my gratitude to Dr. J. Baker, Mr. D. L. Ray, and Mr. R. Lepri for their continuous aid and interest.

I wish also to express my thanks to the staff of the Department of Plant and Soil Sciences for their continuous help.

TABLE OF CONTENTS

NOMENCLATURE

H	Initial piezometric surface, ft.
h	Height of the piezometric surface at a certain time and a certain distance, ft.
h_w	Height of the piezometric surface at the well, ft.
K	Permeability gal./day sq. ft.
K_r	Relative permeability.
L	Distance from the center of the well to the outlet chamber, ft.
Q	Rate of outflow, gpm.
r	Distance from the center of the well, ft.
r_w	Radius of the well, ft.
SS	Steady state.
CRS	Complete radial system.
TC	Temperature correction.

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

Understanding the physical and hydraulic properties of soils is of prime interest to workers in fields such as soil science, engineering, and hydrology. Sound information concerning factors which contribute to the deterioration of these properties is essential for agricultural practices, water resources development, and environmental quality management.

The movement of water into and within the soil has been extensively studied in the past based on purely theoretical assumptions such as constant geometry of soil pores, and a moving fluid having constant properties such as density, surface tension, viscosity, and incompressibility. Results based on these studies have limited application in actual engineering and agricultural practice because the percolating water usually contains many different kinds of impurities such as suspended fine particles due to erosion, and microbiological organisms. These impurities and chemical impurities alter the flow regime and contribute to changes in the physical and chemical properties of the soil.

To investigate the effects of dissolved and suspended materials in water on the physical and hydraulic characteristics of soils and to trace the movement of these impurities within the soil, is of importance and essential to wise management of our water resources. Because impurities in water are of mechanical, chemical and biological nature it is difficult to reproduce such impurities in a laboratory situation. Therefore, it has been decided to use different concentrations and mixtures of sewage and tap water to simulate impurities. This study is tied closely with problems common to agriculture and engineering such as farm waste and sanitary waste disposal.

One important problem is that of combined sewer overflows and their pollutional effects on streams and underground water supplies of the United States. The study reported herein has been combined with a possible solution to the problem of combined sewer overflows proposed by Dr. Donald Dean Adrian and Donald L. Ray of the Department of Civil Engineering, Environmental Engineering Program at the University of Massachusetts. Their solution consists basically of injecting overflow into an unconfined underground formation, and thus, controlling the discharge into the natural water courses. Actual operation of this method is as follows: during the period of potential overflow, wastewater which normally bypasses the treatment plant and is discharged directly into a receiving stream is diverted to flow through a pretreatment unit to remove large objects. It then goes through a high capacity sedimentation unit for removal of settleable materials. The overflow is then discharged by gravity into an underground permeable formation. After a storm has passed and the flow rate reduced to a value capable of being conveyed by the interceptor sewer, the flow in the permeable formation is reversed by pumping the stored overflow from the permeable formation at a rate which does not overload the system. The pumped waste is then treated in a normal manner. This reversal of flow is intended to accomplish two objectives: first, it is assumed that the formation has a certain storage capacity which limits indefinite injection. Therefore, by reversing flow, the level of water in the formation is maintained at a level suitable for the next injection process. Secondly, the reversal of flow serves as a backwash mechanism to unclog interstices which have been filled with solids and thus improve the permeability of the porous material. This helps to redevelop the well for the next overflow injection.

Due to the wide variation in composition and amount of wastewater, several sedimentation units and injection wells would be utilized in a manner so that they can be operated separately or together. The main advantages of this method are: it permits use of the naturally occurring underground formations and, therefore, does not require large land surface areas or high excavating costs; the entire flow can be injected and thus eliminates bypassing into the stream; and injection facilities can be easily located through the drainage area. This method has disadvantages such as: suitable underground formations may not be located where injection is required; the storage characteristics of underground formations which furnish water for agricultural, domestic, or industrial usage may be affected; public health may be endangered by the flow of wastewater through the underground formations; depending on the type of formation, clogging may occur in early stages which will prevent further injections; and the cost of providing these facilities for injection may be so large as to make the system economically unfeasible.

This investigation consisted of the following phases:

1) Development of a mathematical model of the injection process for non-sediment laden water in an isotropic permeable formation. The solutions obtained represent the ideal behavior of the system.
2) Construction of a laboratory model to simulate the injection operation permitting evaluation of the change in the physical characteristics of the soil formation and other factors related to flow such as clogging, change in permeability, discharge rate, and water table elevation as related to distance and time.
3) Comparison of the two models to analyze the effects of sediments on the physical behavior of the system.

LITERATURE REVIEW

The changes in physical properties of soil due to injection of wastewater are mainly a result of the removal of sediments in the wastewater by the soil, usually termed clogging. The physical and hydraulic properties of soils which are severely altered by clogging of the soil pores are: the infiltration rate which is defined as the quantity of injected fluid which will flow through the soil in a unit time; the permeability which expresses the rate of flow through a unit cross sectional area of the soil in the presence of a unit potential gradient; and the water table elevation (piezometric head) inside the soil formation.

A soil may become clogged as a result of mechanical, chemical, and biological processes (19). Mechanical clogging occurs as a result of entrapment of suspended solids within the soil which become lodged in pores. The degree of mechanical clogging depends on the size and gradation of the soil particles, and the velocity of the flow. A task group of the American Water Works Association (26) has concluded that air bubbles, which may be released from the injected water due to the difference in temperature of the injected water and the ground water, can also cause mechanical clogging. Bliss and Johnson (8) and Schief and Johnson (22) have conducted infiltration studies and concluded that clogging of the soil pores is mainly due to microbial growth and the accumulation of undissolved gas.

Chemical factors which induce clogging depend on changes in soil chemical properties caused by wastewater. These changes occur because of the chemical interaction between dissolved materials and clay surfaces. Clogging may be caused by deflocculation of aggregates by sodium present in the wastewater. The settlement of fine particles released from aggregates causes a reduction in the porosity of the soil. Winneberger, *et al* (30) observed that ferrous sulfide, which is formed as a result of anaerobic decomposition of organic matter, contributed to the clogging process. Under aerobic conditions, ferrous sulfide may be readily oxidized to the soluble sulfate form which is washed out. However, Thomas *et al* (27) showed that the infiltration rate remained the same after the sulfide was oxidized by the introduction of aerobic conditions. This indicates that the anaerobic condition, not the sulfide, is the cause of clogging. Changes in the chemical composition of the pore gas plays a major part in the deterioration of the physical properties of the soil by enhancing the clogging process. Winneberger (30), Jones and Taylor (12), and Thomas, *et al* (28) have shown a rapid decrease in oxygen and a rapid increase in carbon dioxide concentration in the pore gas following sewage water injection. This leads to rapid clogging as indicated by a decrease in the infiltration rate.

Biological factors are important in altering the physical properties of soils following the injection of wastewater. Clogging usually occurs as a result of biological growth which is enhanced by inorganic and organic materials in the injected fluid. Allison (2) found that the low permeability caused by clogging was mainly due to microbial activity caused by the disintegration of soil aggregates due to prolonged submergence by water. Martin (17) and McCalla (16) pointed out that microbial cells, their synthesized products, and slimes and polysaccharides are the main cause of biological clogging of soil pores. In addition, Martin (18) concluded that dispersion due to the attack of microorganisms on organic material, which binds soil into aggregates, may also be a factor in changing soil physical properties.

Biological clogging has been found to be closely correlated with polysaccharide and polyuronide production. Studies by Amramy (3) and Mitchell and Nevo (20) showed that under aerobic conditions, polysaccharides accumulated during the clogging process in the soil pores. Duguid and Wilkinson (11) and Lynch (15) found that a surplus of carbon and nitrogen in the soil caused a massive production of polysaccharides. Avnimelech and Nevo (4) reported that clogging was more highly correlated with polyuronide concentration than with polysaccharide concentration.

The movement of bacteria in the soil is associated with the movement of the injection fluid. Chemicals in solution have been found to travel greater distances than bacteria. Two investigations by the University of California and California State Water Quality Control Board (21) and (10) found a reduction from 110,000 coliforms/100 mls to 40,000/100 mls as water traveled three feet within the soil. The soluble products of biogradation may be expected to travel with the injected fluid once the capacity of the soil to adsorb them has been satisfied. Increasing the degree of aggregation in the soil delays the progress of clogging of the soil pores due to the decrease of suspended solids being strained at the soil particles.

Description of Subsurface Disposal Methods

The use of underground formations for the disposal of waste is an old practice. Private on-site disposal systems such as septic tanks have been used for many years in unsewered residential areas. Deep well disposal of highly toxic and non-degradable industrial waste is becoming increasingly popular.

Generally, three methods are used for the disposal of wastewater in underground formations, namely, surface spreading, recharge basins or pits, and injection wells. The geological and hydraulic characteristics present in an area are important factors in selecting one of these methods.

Surface spreading methods involve flooding a ground surface with wastewater and allowing it to percolate through the soil to the ground water. A major problem of this method is to maintain a high infiltration rate (25).

A basin, trench or pit is used for subsurface disposal in areas where a shallow impermeable layer exists and restricts the downward infiltration or in areas where land costs are high, because only a small amount of land is required.

To evaluate the feasibility of using gravity shafts to recharge water from the Cheyenne River into water bearing deposits located beneath 50-150 feet of clay and silt, Skodje (23) conducted laboratory tests using 4 inch inside diameter (ID) and 4 foot high model gravity shafts in which each shaft was filled with a different size of uniformly graded sand. He found that because most clogging occurred within the top 3 inches of the shaft, the recharge rate could be maintained by removing the clogged layer periodically and replacing it with clean sand.

Injection wells have been most commonly used in deep impermeable formations. The methods used to construct injection wells are quite similar to those used to construct pumping wells (26). The capacity of injection wells is usually less than their specific yield as pumping wells. The major advantages of injection wells are that only a small amount of land is required and that they may be placed on the right-of-ways of public roads. Their major disadvantage is that a degree of advanced treatment is normally required to prevent clogging. Corrosion problems, encountered when using injection wells are reduced by using cement, asbestos and stainless steel casings.

Several subsurface injection projects have been conducted in the United States. It is worthwhile to review some of their methods and findings. The Richmond Project conducted by the University of California (9) was primarily directed toward investigating the movement of bacterial and chemical pollutants in water percolating through soil above the ground water table, and to determine if treated domestic sewage would have any adverse effects on the soil. A 12 inch injection well was bored to penetrate a 5 foot thick confined formation located approximately 100 feet underground. The original permeability was found to be approximately 1,400 gal/sq.ft./day. Primary settled sewage was diluted with fresh water and injected into the formation at a rate of 32 gpm for a period of 41 days. The injected liquid had a suspended solids content of 4 ppm, and a BOD of 3.3 ppm and a coliform organisms content of 1 million per 100 ml. It was found that bacteria traveled a maximum distance of about 100 feet in the direction of the ground water movement, whereas chemical pollutants traveled both farther and faster. The investigators found that suspended and colloidal solids were the major causes of clogging. Periodic pumping for short periods of time was found to be effective in redeveloping the well.

The Orange County Water District in California (29) plans to construct a barrier to sea water intrusion, with one element of the barrier to be a pressure ridge created by a line of injection wells about 4 miles inland from the coastline. One possible source of water is reclaimed wastewater. After 7 months of injection of wastewater, the following conclusions were reached: (a) treated trickling filter effluent is injectable and did not cause excessive well clogging, (b) coliform bacteria appeared sporadically 100 feet from the injection well. Human intestinal viruses were not found in the injection water or in any of the observation wells, (c) many chemical constituents do not move conservatively in the injected water. Hardness and alkalinity increase and ammonia and oxygen demanding materials are significantly reduced by travel in the confined formation, (d) some highly resistant soluble organic materials, which cause the odor and taste in the injected reclaimed water, were not sufficiently removed or altered by movement through 545 feet of the confined formation.

Wastewater reclamation using artificial recharge techniques has been practiced for decades on Long Island, New York (7) where wastewater is leached into the soil from cesspools and septic tank systems. The availability of wastewater effluents for artificial recharge to the ground water reservoir offers a means of supplementing naturally available water supply and preventing salt water intrusion into the fresh ground water supplies. At present, there are more than 1,000 recharge wells returning used ground water back to the underground reservoirs. New York State regulations prohibit the construction of new industrial wells with capacities exceeding 69.4 gpm unless the water is returned in an uncontaminated condition to the ground water. Tests are now being conducted to determine the feasibility of constructing a line of "barrier-injection" wells across the entire south shore of Nassau County. Initially 27 mgd of highly treated wastewater will be injected into a series of carefully spaced wells in the underground formation. The purpose of these tests is to obtain information regarding the physical and chemical factors that control the rates and injection pressure at which the treated effluent can be injected.

The U.S. Geological Survey has conducted an investigation of the fundamental principles pertaining to artificial recharge of ground water reservoirs in alluvial deposits through wells. The Grand Prairie region of Arkansas was selected as a site for this investigation (24). The major problem encountered during this investigation was the clogging of wells and the formations. The main causes of clogging were listed as: (a) air entrapment, (b) suspended solids, (c) microorganisms in the recharge water, (d) precipitates caused by chemical reactions between the ground water, formation and recharge water, (e) temperature differences between the injected water and the warmer native ground water and, (f) weather conditions.

Injection of reclaimed wastewater into the recharge well constituted the second major phase of a reclamation study conducted by the Los Angeles County Flood Control District (13) at the Hyperion waste treatment plant. An 8 inch diameter steel casing was placed to a depth of 140 feet below ground surface and perforated between 98 and 132 feet within the confined silverado aquifer. The well was packed with a 2-4 inch gravel blanket. Reclaimed water was chlorinated and supplied to the recharge well by gravity flow under sufficient head to accomplish injection at a rate of .3 cfs. It took the system a few days to maintain a constant head and any increase in the head that was recorded afterward was attributed to clogging, which was caused mostly by the deposition of suspended solids. However, effluent containing as much as 6 percent solids could be tolerated if the solids were of suitable particle size, fully oxidized and contained a minimum amount of inert clay or silt.

Geological Aspects of Injection Wells

In determining the feasibility of a recharge operation in any area, it is important to investigate the geological features of the area. These usually include evaluation of the following properties:

a) The depth and size of the underground formation.

b) The vertical and horizontal permeability of the underground formation.

c) The direction of the ground water movement.

d) The location of existing wells in the area.

e) The location of alluvial fans or wide river valleys in the area.

The recharge rate of a particular site is dependent on the permeability of the formation and where the geological structure of a formation consists of several layers of different permeabilities, the formation with the lowest permeability determines the maximum recharge rate.

LABORATORY SIMULATION OF THE INJECTION PROCESS

In order to investigate the effects of injecting different concentrations of sewage on the physical characteristics of the unconfined formation, a laboratory simulator was constructed. Varying the duration of injection was necessary in order to obtain better simulation of rainstorm durations. Data obtained using the laboratory simulator is used to evaluate the technical practicality of the proposed system in solving combined sewer overflow problems.

Description and Operation of the Simulator

If water is injected in an unconfined underground formation while maintaining a constant gravity head at the injection well, it flows radially away from the point of injection with the velocity decreasing with distance. If it is assumed that flow is symmetrical about the injection well, then it is unnecessary to model the entire flow area. A sector of the circle of influence can be selected and still preserve the characteristics of the total injection flow regime. This was done using 5/8" plexiglas to construct a 15° pie-shaped model injection well, 6 feet long and 4 feet deep. A schematic diagram showing the main parts and dimensions of the model is shown in Figure 1.

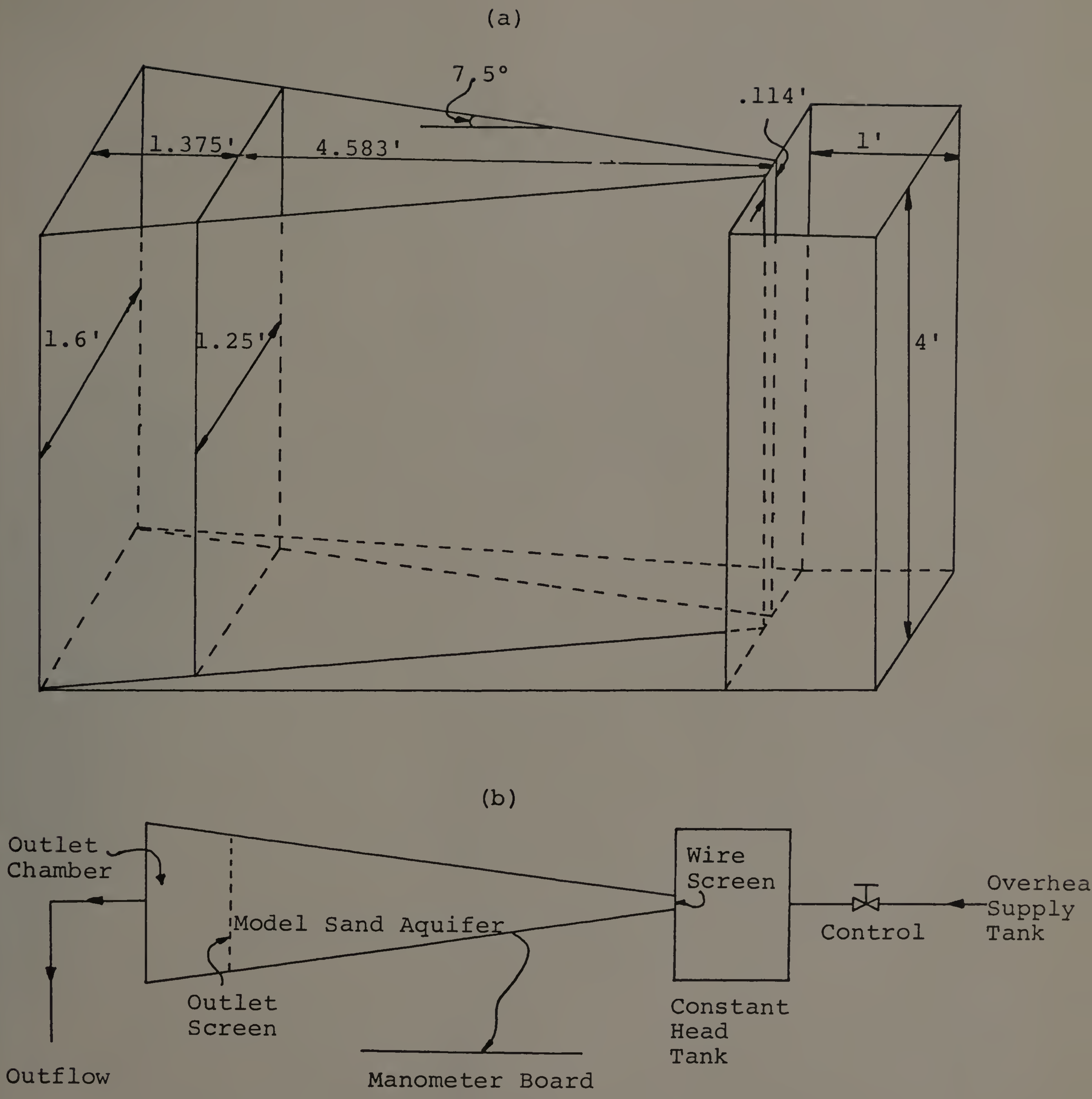


FIGURE 1

SCHEMATIC DIAGRAM SHOWING THE MAIN PARTS AND DIMENSIONS OF THE SIMULATOR. (a) Side view, (b) Top view

Twenty manometer taps at the bottom of one of the side walls were connected to a manometer board using rubber tubing. Distances or these manometer taps to the center of the well are given in Table 1.

TABLE 1

DISTANCES OF THE MANOMETER TAPS FROM THE CENTER OF THE WELL

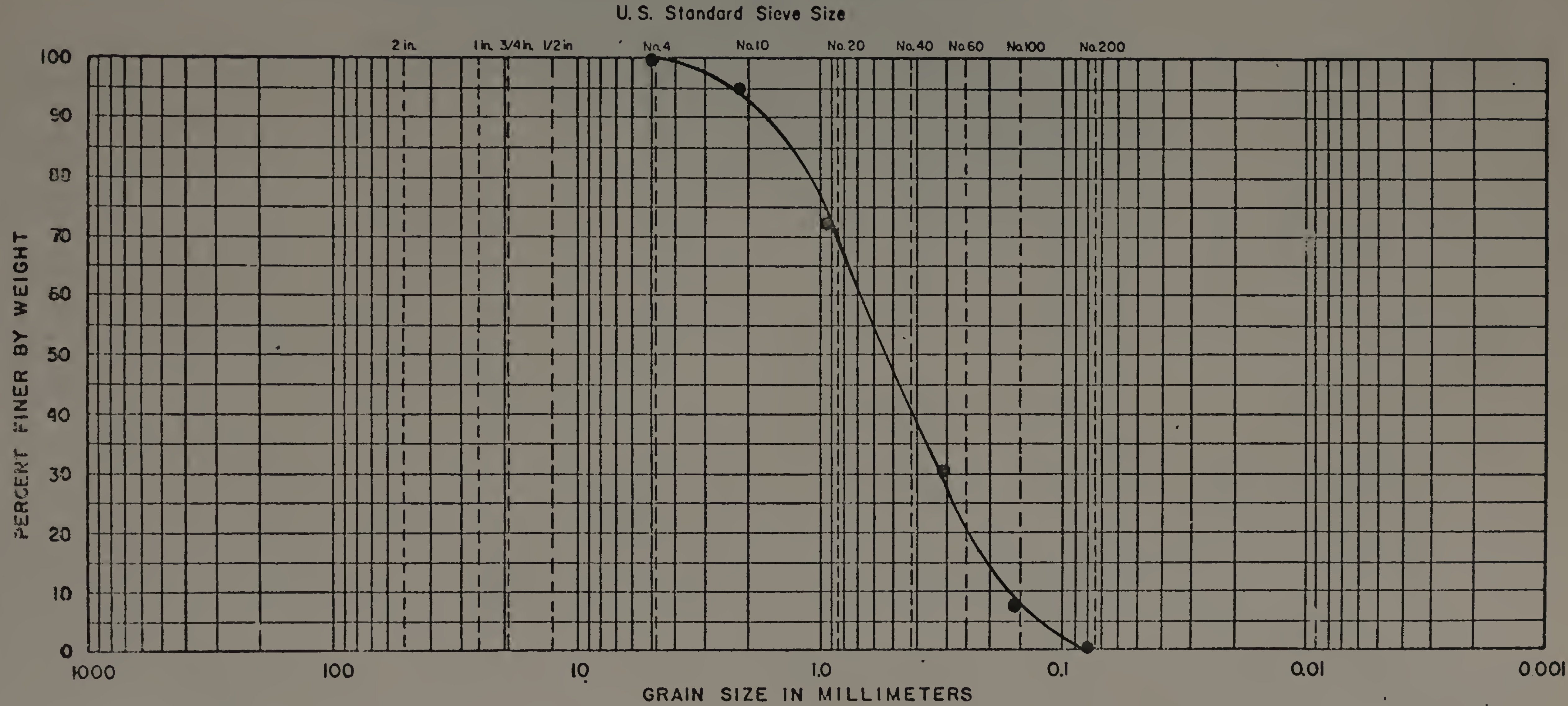
Manometer Number	Distance in Feet
1	.547
2	.670
3	.794
4	.918
5	1.041
6	1.165
7	1.289
8	1.412
9	1.536
10	1.660
11	1.783
12	1.907
13	2.153
14	2.401
15	2.650
16	2.899
17	3.393
18	3.888
19	4.383
20	4.878

Constant heads were maintained at the well and the outflow chambers using overflow tap control mechanisms. Sunderland Mason sand having a size distribution pattern and properties as shown in Figure 2 was used to simulate an underground formation.

The following operational sequence for the simulator was carried out in each test run. The simulator was filled with dry sand and compacted to give maximum possible density. To prevent the entrapment of air bubbles, the sand was saturated from the base with tap water. The sand was then washed to reduce the amount of fine sediments by first maintaining a constant drawdown of .862 feet at the well and a constant inflow level of 2.792 feet for approximately three to four hours and then reversing the flow direction by maintaining a constant level of .825 feet at the outflow chamber and 2.792 feet at the well for approximately one to two hours. Once the sand had been washed in both directions, the water level was maintained at .825 feet in both the well and the outflow chamber.

To initiate an experimental run, the water level in the well was suddenly increased to 2.792 feet and kept constant while the level in the outflow chamber was kept constant at .825 feet. Consequently, water flowed from the well to the outflow chamber and the piezometric surface inside the sand increased until steady flow was established. The piezometric surface was then recorded by reading the manometers and the

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT or CLAY.
	COARSE	FINE	COARSE	MEDIUM	FINE	

UNIFIED SOIL CLASSIFICATION SYSTEM, CORPS OF ENGINEERS, U.S. ARMY

Uniformity Coefficient = 66/17 ≃ 4
Specific Gravity = 2.68
Permeability (Falling Head, Void Ratio = .7) ≃ .1 ft/min

The sand is identified as an SP soil in the unified soil classification system.

FIGURE 2

PROJECT Sunderland Mason Sand Brown, Mostly Quartz

FILE NO. ______ DATE 20 Jan 68

flow rate was measured by recording the time required to collect samples of known volume. The temperature was recorded at both the inlet and outlet sides of the simulator. From this, the permeability of the soil material, as it actually existed in the simulator, was determined using the procedure outlined in Appendix A. Plots of the piezometric surface at different times were used to compare changes in the piezometric surface as a function of time, caused by clogging of small pores by suspended materials in subsequent runs.

The wastewater used for injection was a mixture of tap water and settled sanitary waste obtained from the Amherst Sewage Treatment Plant, Amherst, Massachusetts. Four basic step-wise injection plans were adopted to simulate the concentration changes likely to occur during the time of a combined sewer overflow (Figure 3). When following plans 1, 2A, and 3, each injection step lasted for 1.5 hours, whereas in plan 2B each step lasted for 45 minutes.

Plan 1 represents an overflow where the concentration of pollutants increases with time, while plans 2A and 2B represent an overflow where the concentration of pollutants decreases with time for long (2A) and short (2B) injection periods. Plan 3 simulates first an increase in concentration of suspended materials for the first 4 hours of a rainstorm and then a decrease for the last 5 hours. To insure a better understanding of the effects of the injection process, test runs following plans 1, 2A, and 3 were made in duplicate.

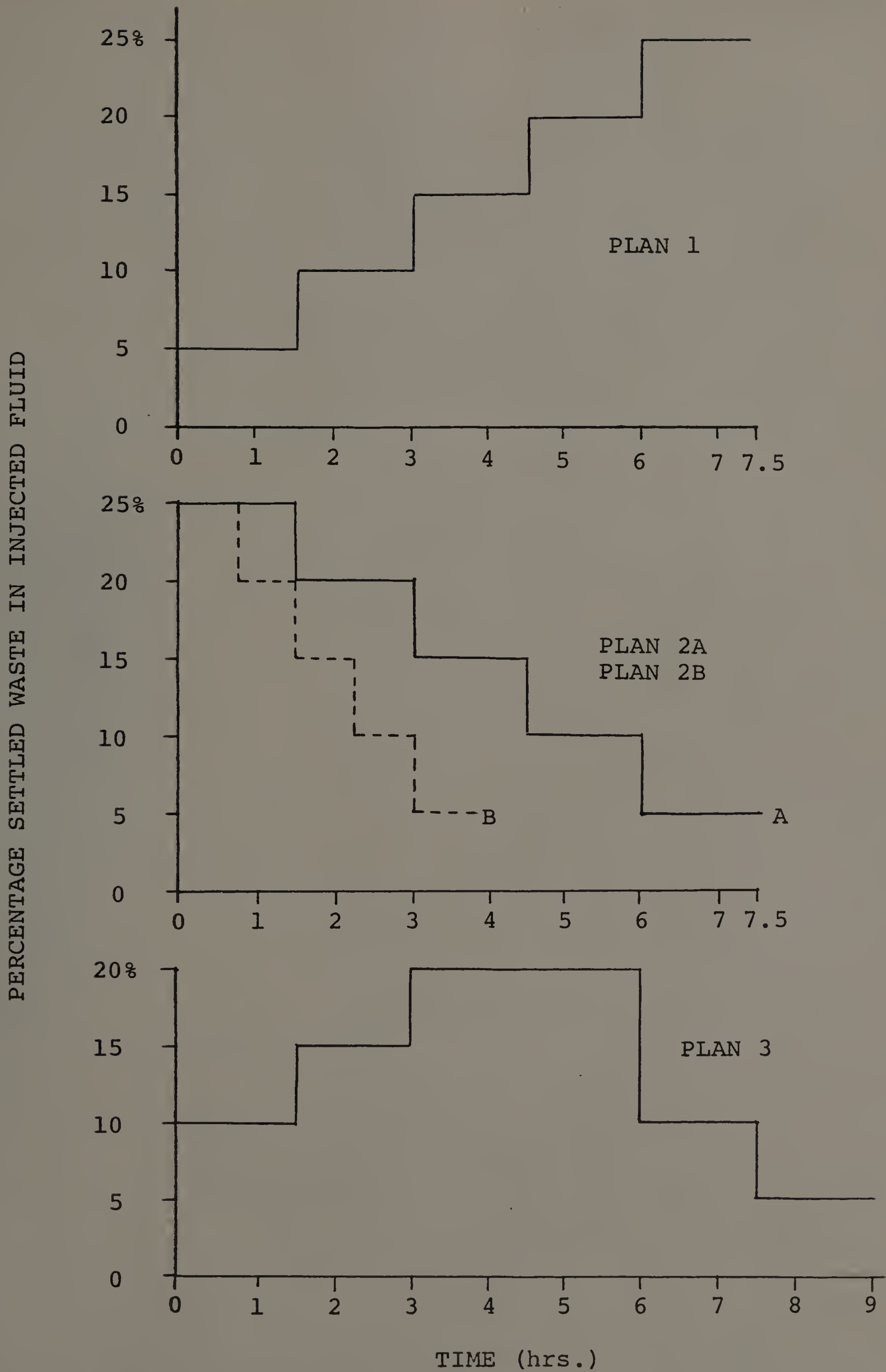


FIGURE 3

CHANGES IN SEWAGE CONCENTRATION IN THE INJECTION FLUID DURING EXPERIMENTAL RUNS FOLLOWING PLANS 1, 2A, 2B and 3

When following plans 1, 2A, and 3, the concentration at the well was equated to the initial concentration of the plan under investigation and the sewage material was injected through the formation by keeping a constant level of 2.792 feet at the well and .825 feet at the outlet chamber. The piezometric surface started to drop and the rate of outflow decreased because of clogging of pores by sediments in the injection sewage mixture thus causing a decrease in permeability.

The piezometric surface, the flow rate, and the temperature of the sewage mixture at both the well and the outlet chamber were recorded every hour during runs following plans 1, 2A, and 3. From this information, the changes in permeability and position of the piezometric surface were calculated as a function of time. Because permeability depends on the viscosity of the liquid, which in turn depends on the temperature, temperature measurements were made and permeability coefficients were standardized at 20°C.

To further simulate field conditions, two injection schemes were carried out. The first was to investigate the effect of short duration rainstorms on the injection process by following injection plan 2B for three consecutive runs without changing the sand in the simulator. At the end of each run, the system was allowed to drain overnight (eight to nine hours) and pumped for 30 minutes the next morning with tap water and by keeping a constant level of 2.792 feet at the outlet chamber and .862 feet at the well. The purpose of pumping was to drive out some of the sediments which had settled in the pores during the injection process, and, therefore, help to restore the permeability of the formation.

Because the wetted zone, which was established in the formation by the pumping process, does not cover the zone which was previously wetted by the injection process, 3 inches of tap water was applied at the top of the formation to backwash the volume of the formation which was not affected by pumping (Figure 4). In actual field operation this might be accomplished by constructing a metal ring, 3-4 feet in diameter and 6 inches deep, around the well and filling it with tap water during each pumping cycle to allow vertical infiltration.

Scheme 2 was planned to investigate the effect of periodic pumping over an extended period of injection. This simulated having multiple wells separated by distances greater than their radius of influence. Diverting the flow from one injection well to another enables the injection process to be interrupted and the pumping cycle to be conducted. This consisted of following injection plan 2A for 3 consecutive runs with 15 minutes of pumping every 3 hours. At the end of each run the system was pumped for 15 minutes, left over-night, and then pumped for 30 minutes the following morning.

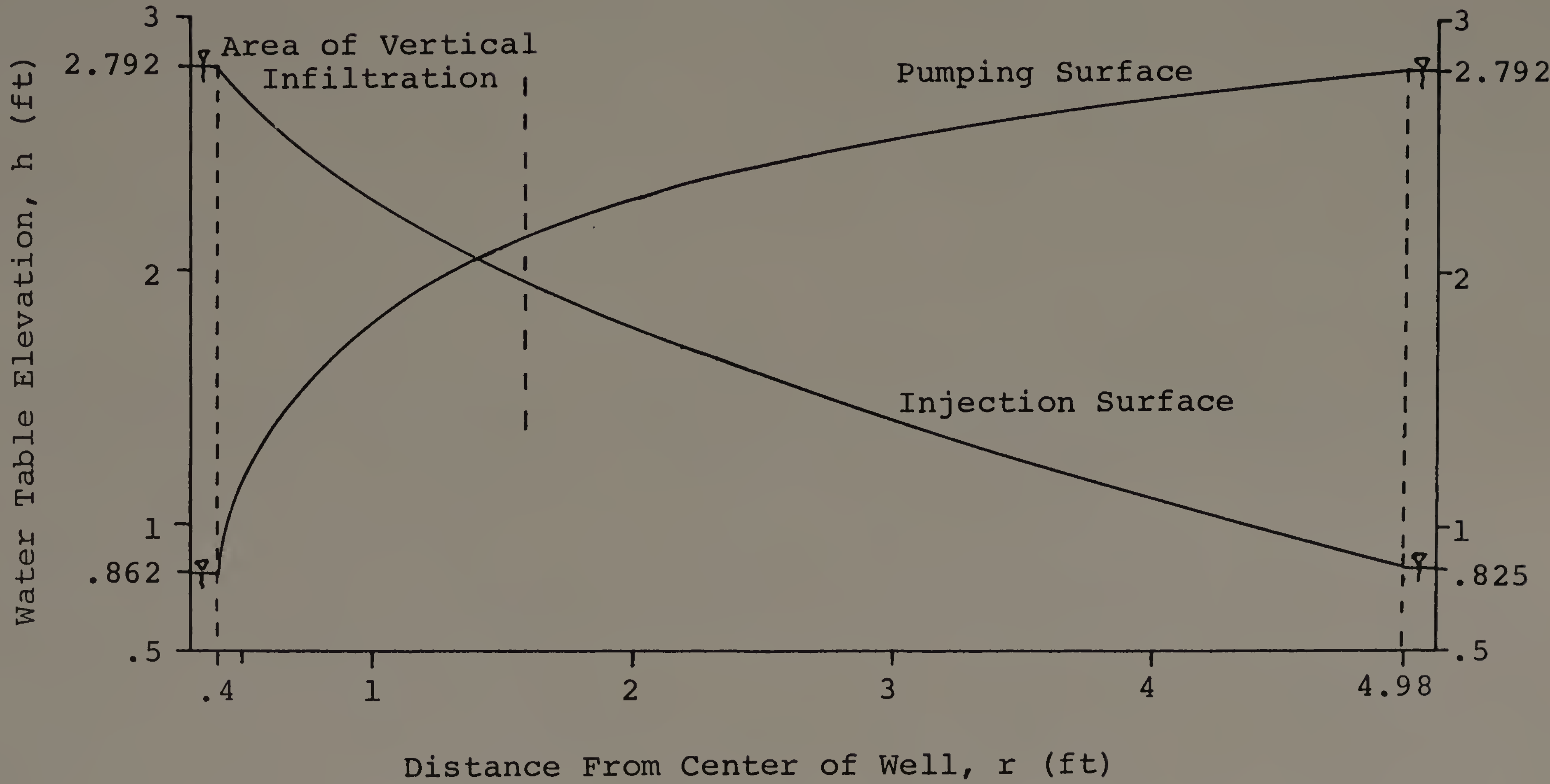


FIGURE 4

THEORETICAL INJECTION AND PUMPING PIEZOMETRIC SURFACES

Theoretical Considerations

It was previously mentioned that the system was brought to a steady state before commencement of injection for each test run. If it is assumed that the flow is through a homogeneous, isotropic, semi-infinite saturated medium, which overlays an impermeable horizontal layer, and that Darcy's and Dupuit-Forcheimer's assumptions hold, then the radial flow from the well can be described by the following one dimensional ordinary differential equation:

$$\frac{1}{r}\frac{d}{dr}\left(r\frac{dh^2}{dr}\right) = 0 \qquad (1)$$

subject to the boundary conditions:

$$h = h_w \qquad \text{for } r = r_w \qquad (2)$$

and

$$r\frac{dh^2}{dr} = -\frac{Q}{\pi K} \qquad (3)$$

Two successive integrations of (1) render:

$$h^2 = h_w{}^2 - \frac{Q}{\pi K}\ln\left(\frac{r}{r_w}\right) \qquad (4)$$

A plot showing the theoretical free surface versus r for the simulator is shown in Figure 5.

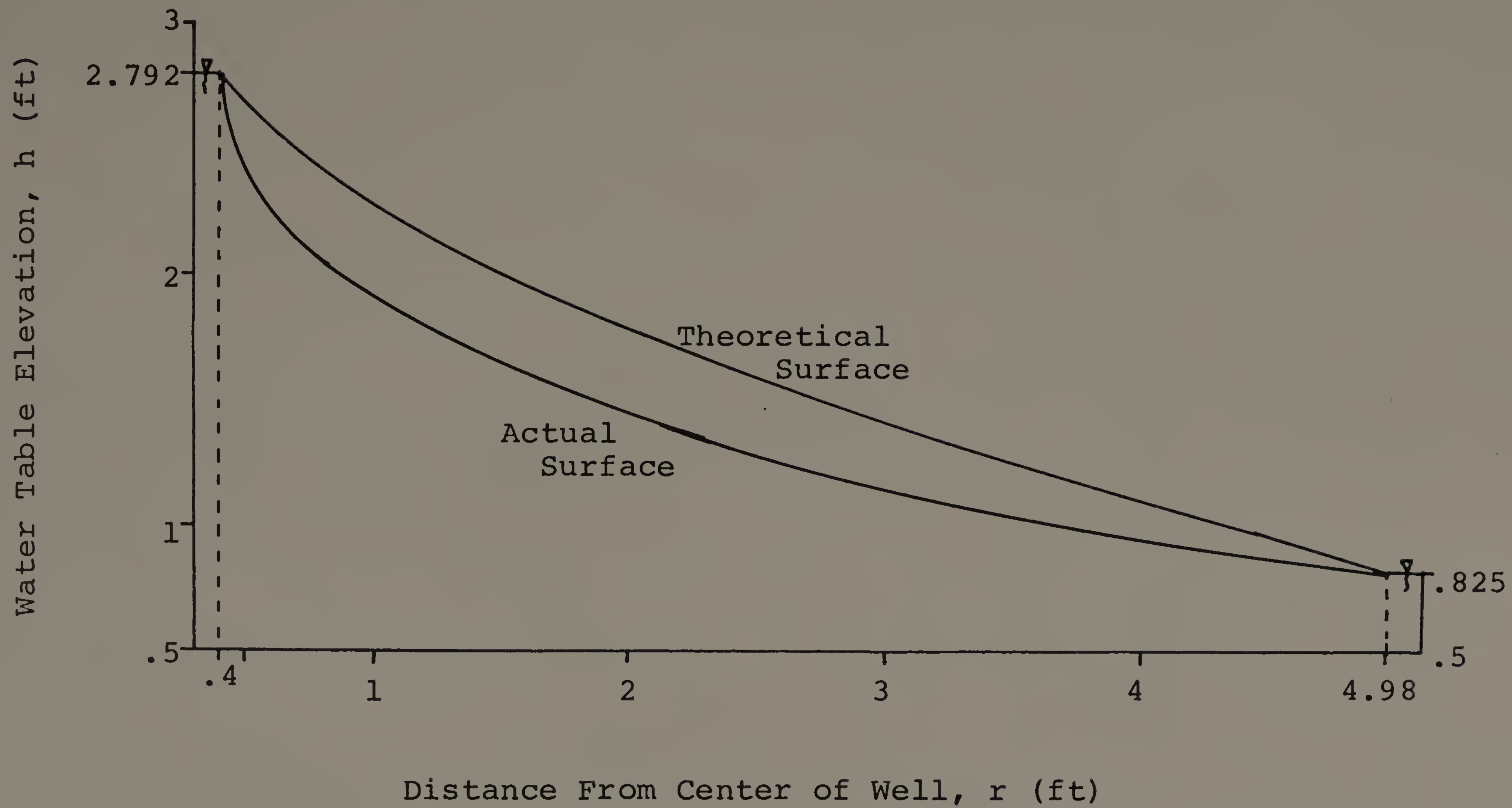


FIGURE 5

ACTUAL AND THEORETICAL PIEZOMETRIC SURFACES

Equation (4) does not consider the existence of a seepage surface above the water level in the outlet chamber. Because of the underlying Dupuit assumptions, equation (4) fails to describe accurately the injection free surface near the well, where the steep curvature of the water table contradicts these assumptions.

RESULTS AND DISCUSSION

The theoretical Dupuit injection piezometric surface was calculated using equation (4). The steady state piezometric surface was obtained experimentally using data from the manometers and tap water at the injection water. Both surfaces are shown in Figure 5.

The experimental surface shows a sudden drop at the formation well interface which does not conform with the shape of the Dupuit injection surface. The differences between the two curves suggests that a boundary entrance head loss exists at the formation interface of the injection well. Sediments in the injection water tended to intensify this loss because of clogging.

Babbitt and Caldwell (5) ascribed the entrance head loss at a pumping well to capillary and surface tension. It is reasonable to assume that the characteristics of the formation material, pore size distribution, well diameter, mass flux flow, temperature, surface tension, kinematic viscosity, and the density of the injected fluid are important factors which may influence the configuration of the piezometric surface. It is also logical that a head loss occurs when water enters a formation just as the head loss occurs as water enters any conduit (6).

Permeability calculations based upon field pumping tests commonly assume a Dupuit surface. This procedure results in an error of calculated permeabilities because actual field surfaces do not conform to a Dupuit surface. A modified procedure used here is based on a least squares method which minimizes the deviation between the Dupuit surface and the experimental surface. Permeabilities were calculated for distances of .719, 1.412, 2.341, and 3.393 feet from the center of the well in the simulator. These permeabilities were then corrected for liquid temperature at 20°C. The procedure is discussed in Appendix A. This procedure proved to be an excellent method to account for the relative decrease in permeability of the wetted zone due to clogging.

A portion of the outflow that was measured from the simulator is due to the flow in the unsaturated zone above the free surface. Because it is difficult to estimate separately the amount of this portion of the outflow, it had to be included in calculating permeability and in the measured outflow.

The chemical and biological composition of overflow varies widely with time and intensity of rainstorms and it depends upon the composition of industrial, domestic and commercial wastewater. It would be valuable if pollutographs showing time changes in concentration of different constituents of sewage water were available. Lepri (14) measured the change in suspended solids, biochemical oxygen demand, and total organic carbon of effluents for the wastewater used in this investigation. Some of his findings are presented in Figure 6. This author measured the density of the sewage water prior to its use in the test runs. Nine samples taken at different times had densities ranging from .990 to .994 gm/cc.

From these two studies it is clear that the chemical composition of sewage water varies appreciably with time which suggests that the chemical composition of the combined sewer overflows varies accordingly. This in turn indicates that the effects of combined sewer overflows on the physical properties of permeable formations will vary depending on the chemical, physical, and biological properties of the overflow at the time of injection. To avoid confusion, a reference schematic diagram showing the different plans, runs, and schemes used in this investigation is presented in Table 2.

The steady state piezometric surface obtained by injecting only tap water and the final piezometric surface at the end of each run for plans 1, 2A, and 3 are shown in Figures 7-12. Data obtained for piezometric surfaces at steady state and at the end of each hour, and at the end of the run is presented in Tables 1, 3, 5, 7, 9, and 11 of Appendix B.

The slope of the piezometric surface increased near the well because sediments caused clogging in the formation. The rate of increase in head loss, as shown by differences between initial and final surfaces, was greatest when plan 2A

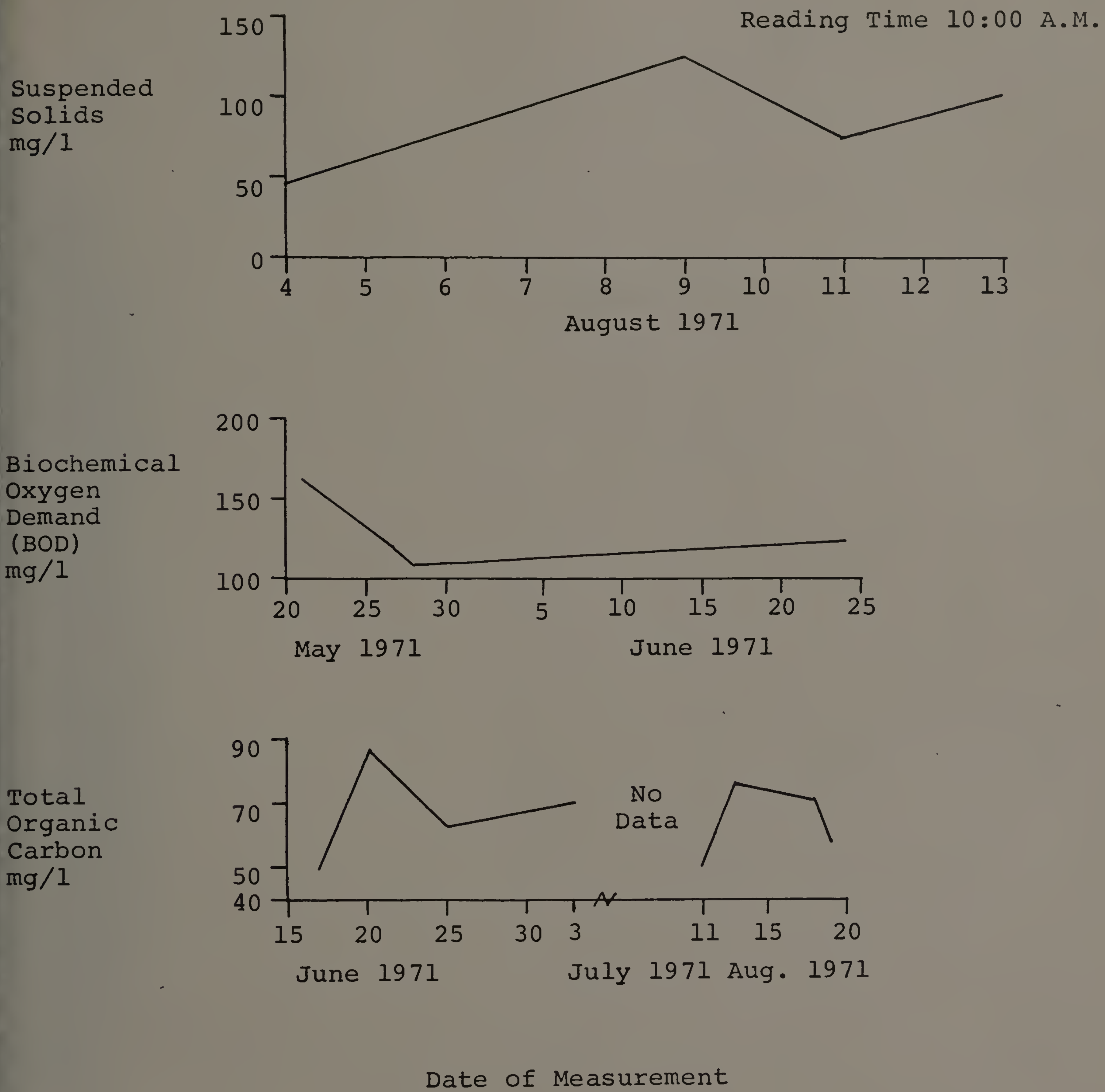


FIGURE 6

CHANGES IN AVERAGE CONCENTRATION OF SEWAGE AT THE AMHERST SEWAGE TREATMENT PLANT

TABLE 2

OUTLINE OF EXPERIMENTAL PROCEDURES FOLLOWED DURING THIS INVESTIGATION

Scheme	Plan	Run	Initial Conc.	Final Conc.	Injection Time Hrs.	At the End of the Run
	1	1	5%	25%	7.5	Change the Sand
		2	5%	25%	7.5	Change the Sand
	2A	1	25%	5%	7.5	Change the Sand
		2	25%	5%	7.5	Change the Sand
	3	1	10%	5%	9	Change the Sand
		2	10%	5%	9	Change the Sand
1	2B	1	25%	5%	3.75	Pumping (30 Min)
		2	25%	5%	3.75	Pumping (30 Min)
		3	25%	5%	3.75	Change the Sand
2	2A	1	25%	5%	7.5	Pumping (15 Min) every 3 Hrs.
		2	25%	5%	7.5	Pumping (15 Min) every 3 Hrs.
		3	25%	5%	7.5	Pumping (15 Min) every 3 Hrs.

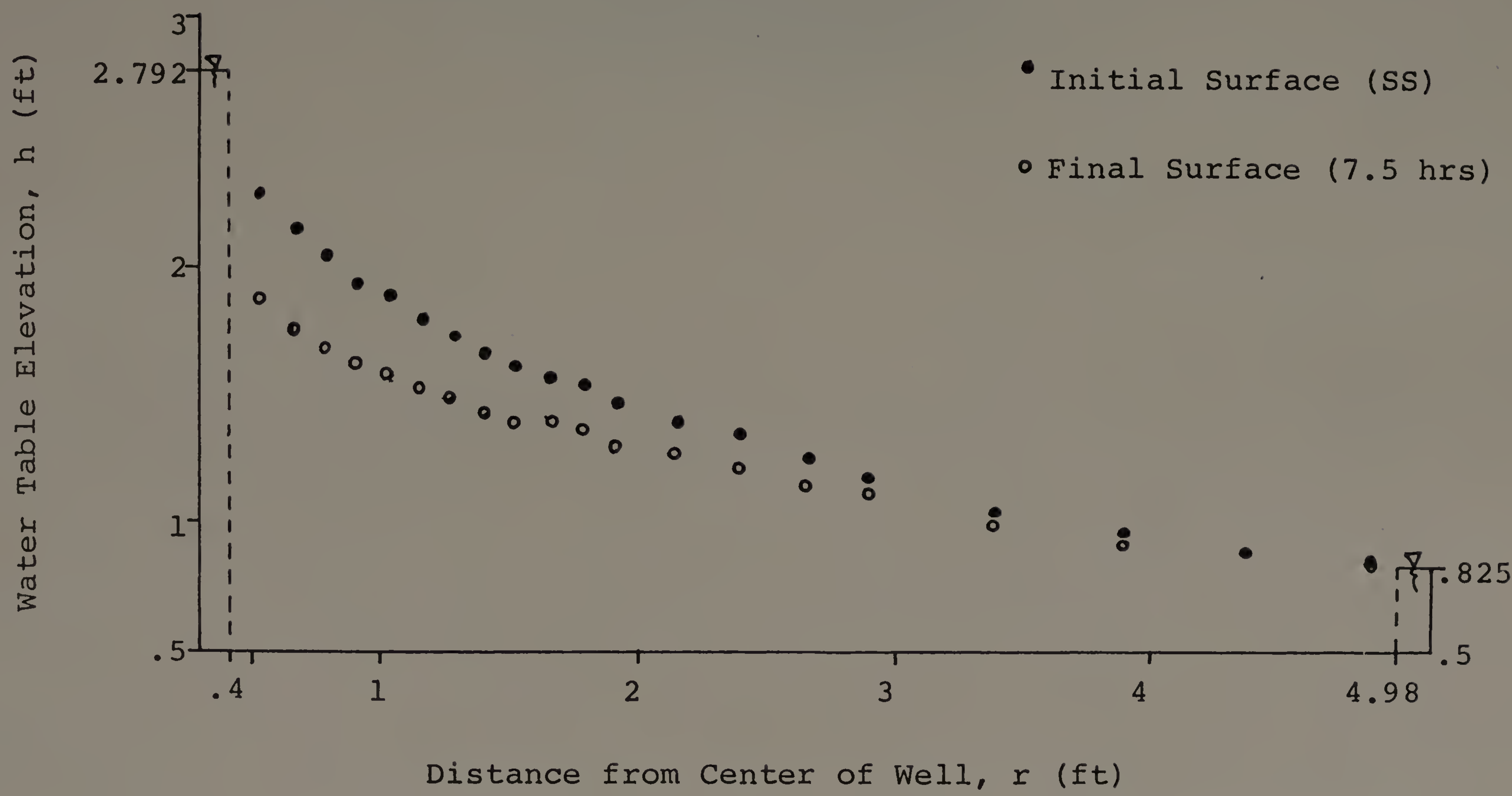


FIGURE 7

INJECTION PIEZOMETRIC SURFACES (RUN 1, PLAN 1)

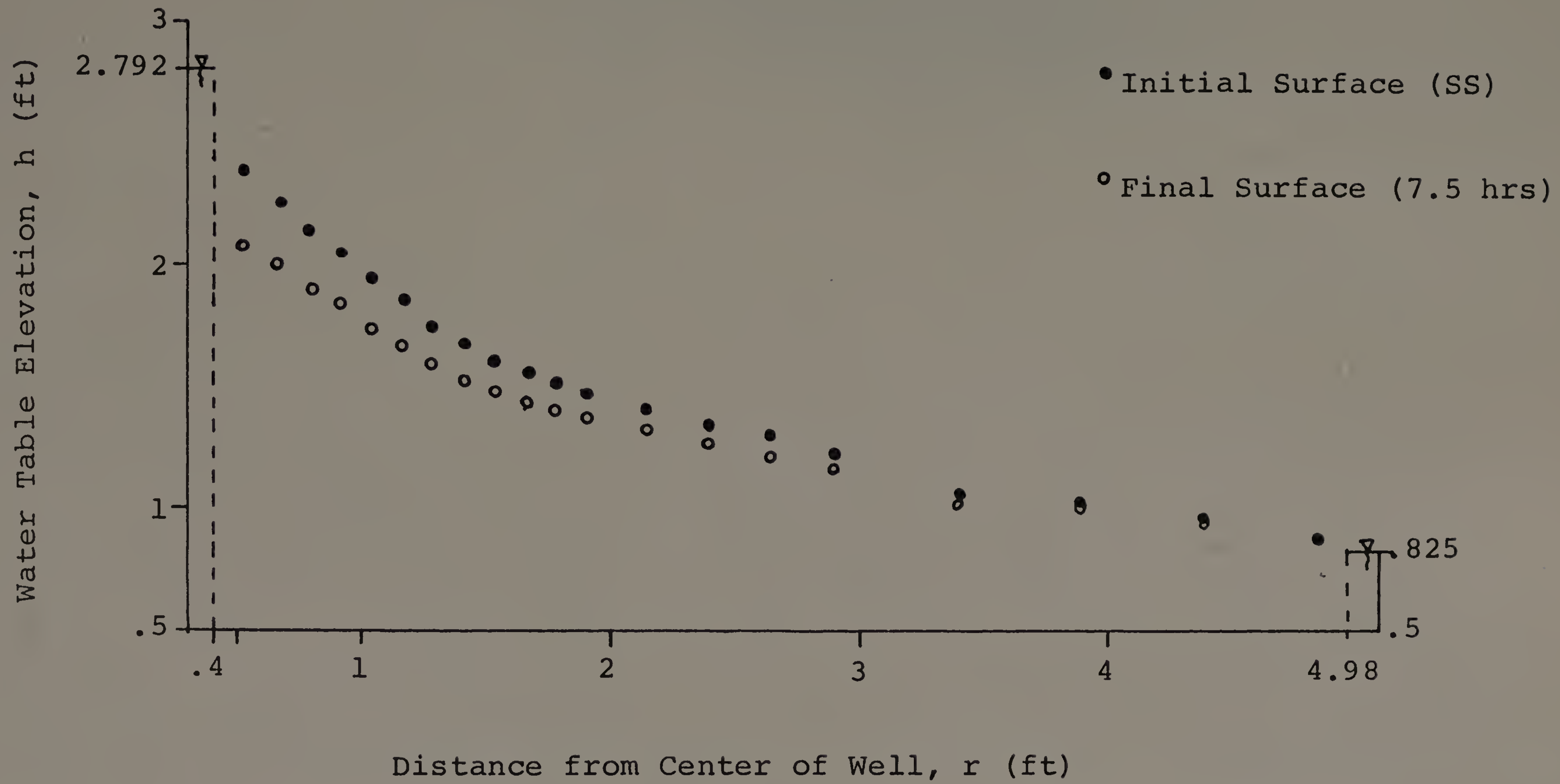


FIGURE 8

INJECTION PIEZOMETRIC SURFACES (RUN 2, PLAN 1)

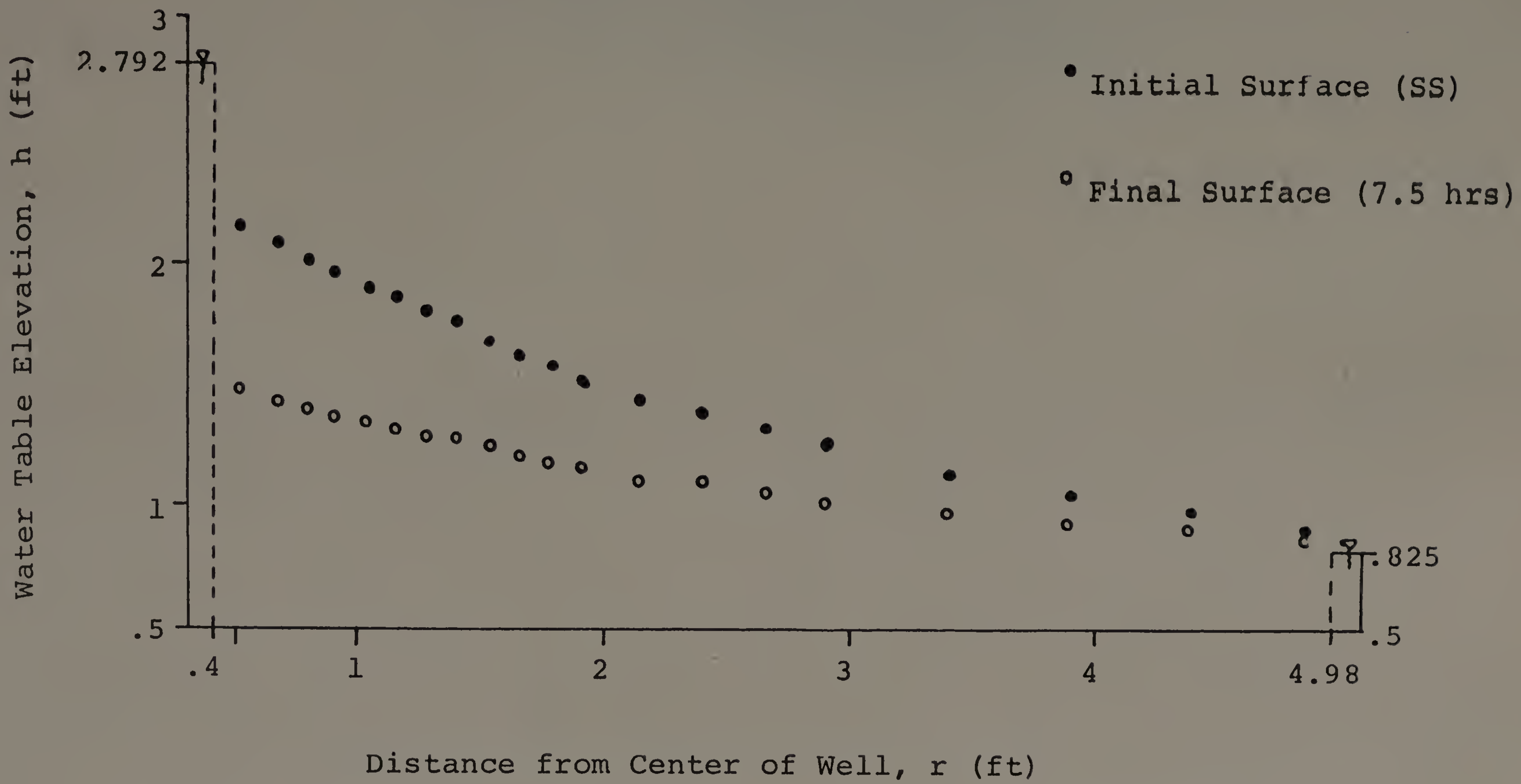


FIGURE 9

INJECTION PIEZOMETRIC SURFACES (RUN 1, PLAN 2A)

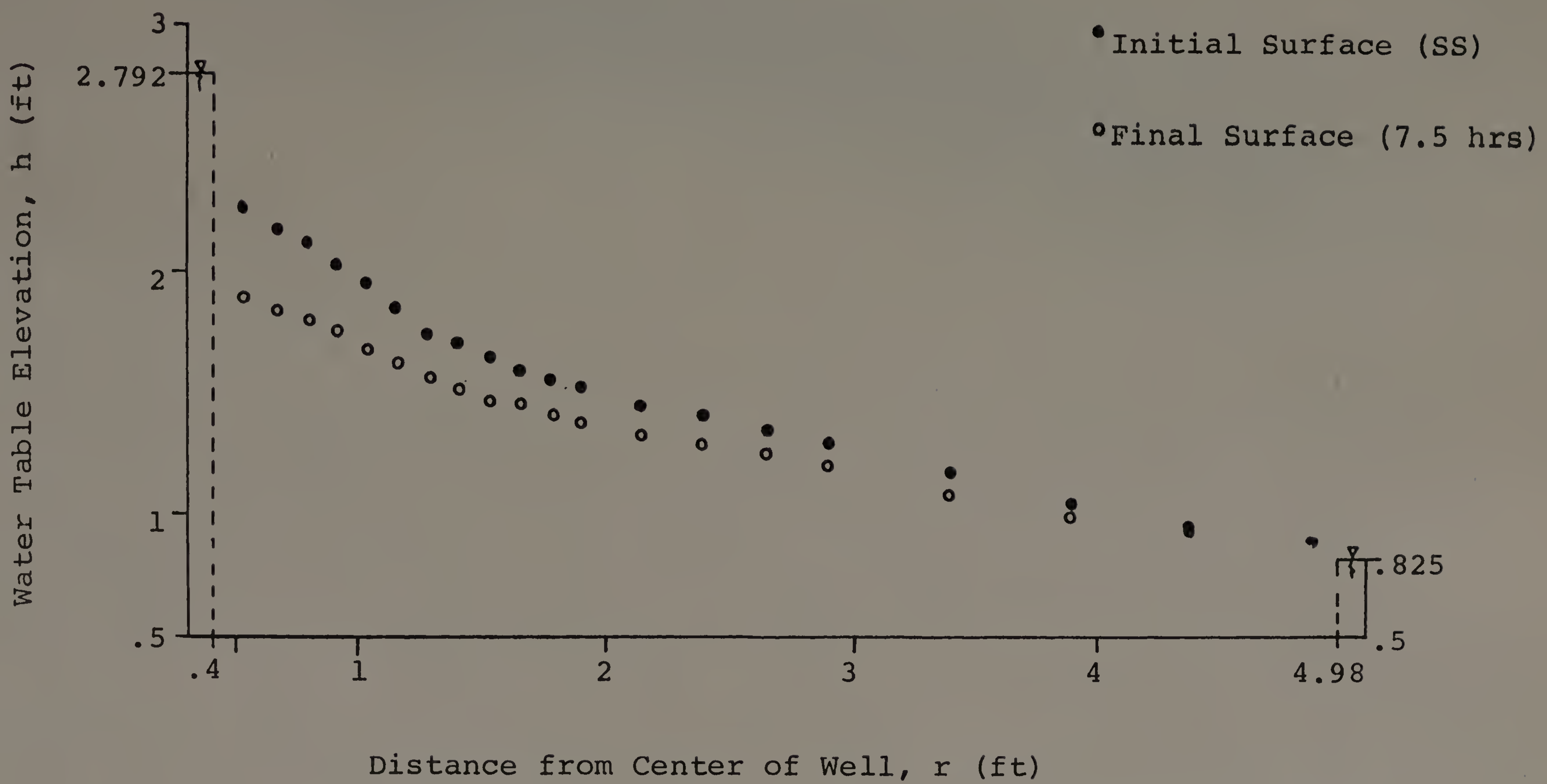


FIGURE 10

INJECTION PIEZOMETRIC SURFACES (RUN 2, PLAN 2A)

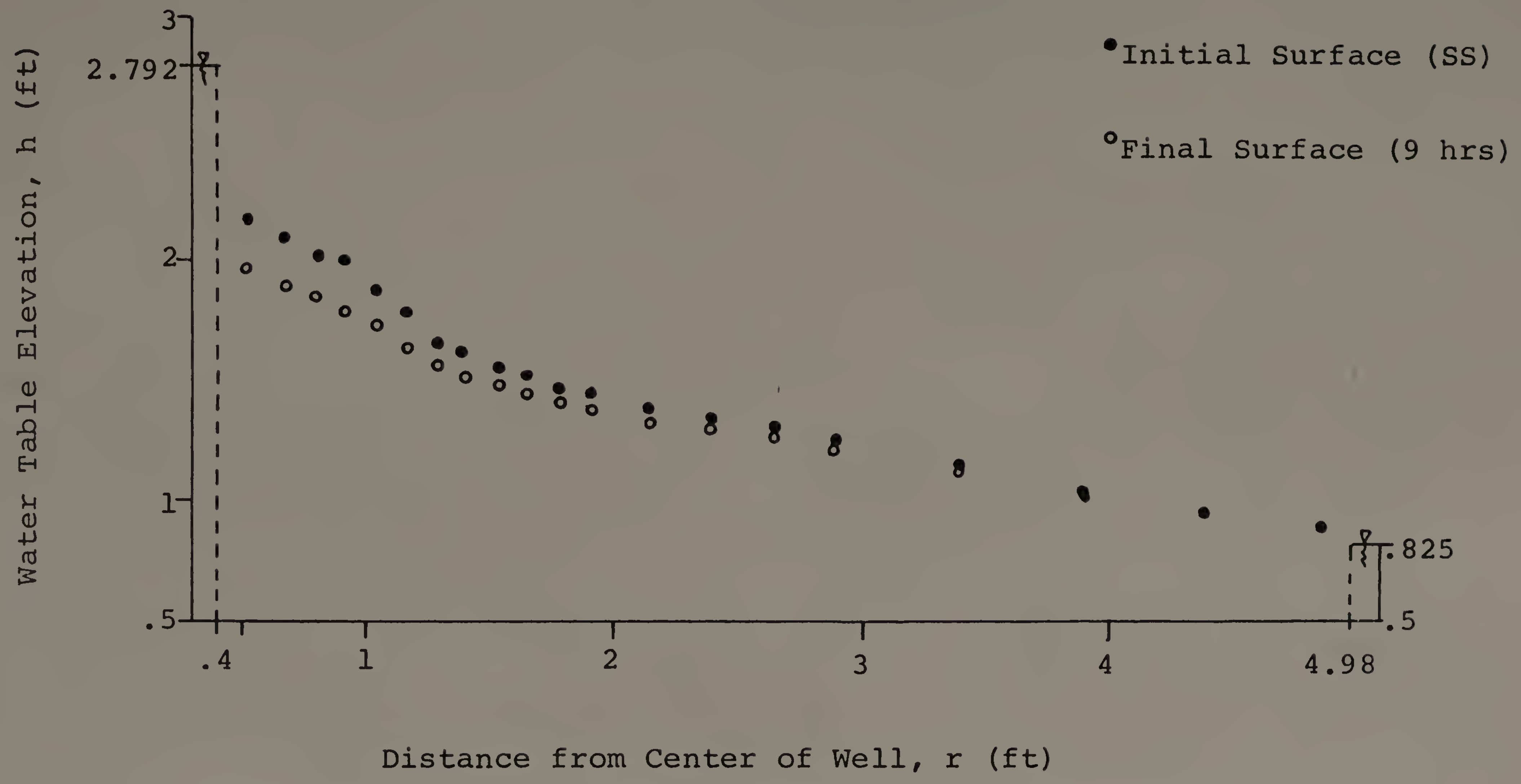


FIGURE 11

INJECTION PIEZOMETRIC SURFACES (RUN 1, PLAN 3)

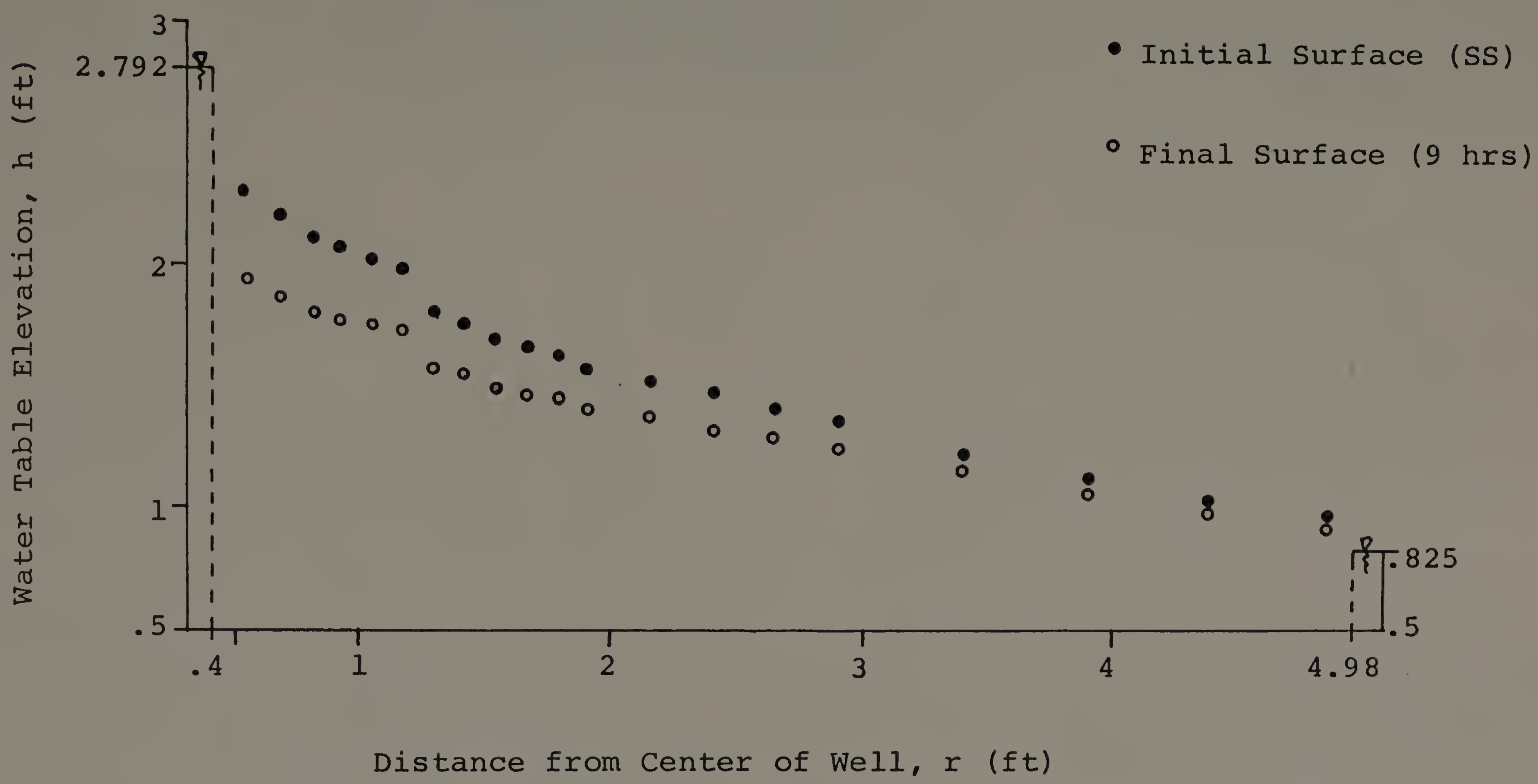


FIGURE 12

INJECTION PIEZOMETRIC SURFACES (RUN 2, PLAN 3)

was followed and the initial concentration of the injected fluid was 25%. This increase was small for plans 1 and 3 at early stages but increased as the concentration of the injected fluid increased. For plans 1 and 3 the greatest head loss occurred in the well screen zone and in the first 2 foot section of the formation, while for the first run following plan 2A, a sharp head loss extended to a distance of about 3 feet away from the well (Figure 9). A smaller head loss occurred during the second run (Figure 10). In all plans the head loss was unnoticeable at a distance of 4.5 feet from the well.

Continuous clogging of the formation, caused by suspended materials in the injected water, resulted in a decrease in the mass flux flow through the formation. This may be attributed to two factors.

a) A decrease in size of flow channels caused by the settlement of sediments in pores, which also caused greater resistance to flow.

b) A drop in the piezometric surface which decreased the amount of material contributing to flow.

Tables 3-8 show the rate of outflow "Q" in gallons per minute (gpm) as computed for a radial system for plans 1, 2A, and 3.

After 1 hour from the beginning of injection, plans 1 and 3 showed an average decrease in Q of 8% compared to Q at steady state. As the injection concentration increased, a 15% decrease in Q was observed for plan 3 after an elapsed

TIME HR.	Q GPM(CRS)
S S	8.349
1.0	8.146
2.0	8.035
3.0	7.940
4.0	7.641
5.0	7.463
6.0	7.172
7.0	6.813
7.5	6.652

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.90	.86	.84	.79	.76
1.412	.96	.94	.93	.87	.85
2.341	.98	.96	.95	.91	.89
3.939	.99	.98	.97	.93	.91

(CONTINUED)

	RELATIVE PERMEABILITY AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.719	.72	.65	.62
1.412	.80	.75	.72
2.341	.85	.80	.78
3.939	.87	.83	.81

TABLE 3

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 1 PLAN 1

TIME HR.	Q GPM(CRS)
S S	7.760
1.0	6.812
2.0	6.479
3.0	6.565
4.0	6.645
5.0	6.597
6.0	6.596
7.0	6.448
7.5	6.393

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.79	.73	.73	.73	.71
1.412	.83	.79	.79	.80	.79
2.341	.85	.81	.81	.82	.82
3.939	.87	.83	.83	.84	.84

(CONTINUED)

	RELATIVE PERMEABILITY AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.719	.69	.66	.65
1.412	.77	.76	.75
2.341	.80	.79	.78
3.939	.82	.81	.81

TABLE 4

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 2 PLAN 1

TIME HR.	Q GPM(CFS)
S.S.	11.117
1.0	9.381
2.0	8.760
3.0	8.209
4.0	7.534
5.0	6.819
6.0	5.984
7.0	5.289
7.5	5.241

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.74	.65	.58	.51	.45
1.412	.79	.71	.65	.58	.51
2.341	.82	.75	.70	.64	.57
3.939	.84	.78	.73	.67	.60

(CONTINUED)

	RELATIVE PERMEABILITY AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.719	.38	.33	.32
1.412	.44	.38	.37
2.341	.49	.43	.42
3.939	.53	.46	.46

TABLE 5

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 1 PLAN 2A

TIME HR.	Q GPM(CFS)
S S	6.474
1.0	5.324
2.0	5.212
3.0	5.208
4.0	5.191
5.0	5.094
6.0	4.985
7.0	4.796
7.5	4.752

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.71	.68	.68	.67	.64
1.412	.78	.74	.77	.76	.73
2.341	.79	.77	.78	.77	.76
3.939	.81	.80	.80	.80	.78

(CONTINUED)

	RELATIVE PERMEABILITY AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.719	.61	.58	.57
1.412	.71	.67	.66
2.341	.74	.71	.70
3.939	.77	.74	.73

TABLE 6

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 2 PLAN 2A

TIME HR.	Q GPM(CRS)
S S	6.132
1.0	5.482
2.0	5.219
3.0	5.180
4.0	5.271
5.0	5.334
6.0	5.215
7.0	5.100
8.0	4.890
9.0	4.795

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.85	.79	.78	.79	.80
1.412	.88	.81	.83	.85	.86
2.341	.87	.83	.82	.84	.85
3.939	.88	.83	.82	.84	.86

(CONTINUED)

	RELATIVE PERMEABILITY AT			
R FT.	6 HR.	7 HR.	8 HR.	9 HR.
.719	.77	.73	.69	.66
1.412	.82	.79	.75	.72
2.341	.84	.81	.77	.75
3.939	.84	.82	.78	.76

TABLE 7

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 1 PLAN 3

TIME HR.	Q GPM(CFS)
S S	6.950
1.0	6.537
2.0	6.387
3.0	6.363
4.0	6.325
5.0	6.199
6.0	5.928
7.0	5.727
8.0	5.479
9.0	5.328

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.90	.85	.82	.80	.76
1.412	.92	.88	.87	.86	.83
2.341	.93	.90	.89	.88	.86
3.939	.93	.90	.90	.90	.88

(CONTINUED)

	RELATIVE PERMEABILITY AT			
R FT.	6 HR.	7 HR.	8 HR.	9 HR.
.719	.69	.64	.60	.56
1.412	.77	.74	.69	.66
2.341	.81	.77	.73	.70
3.939	.65	.80	.76	.73

TABLE 8

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 2 PLAN 3

time of 5 hours, whereas plan 1 showed a decrease of about 12% after 4 hours. At the end of experimental runs following plans 1 and 3 (7.5 and 9 hours), average decreases in Q of 19% and 23% were recorded respectively. Runs following plan 2A resulted in a sharp decrease in the rate of outflow due to the injection of sewage water having higher concentrations (25%). After 1 hour an average decrease in Q of 27% was observed. The outflow measured during run 1 decreased sharply with time and after 4 hours a decrease of 33% was observed and at the end of the run (7.5 hours) Q was only 47% of the steady state value. Run 2, plan 2A showed a smaller decrease in Q at higher concentrations where a decrease of 20% was observed after 4 hours and 27% at the end of the run (7.5 hours). This lower decrease in Q of run 2, plan 2A may be attributed to differences in sewage used in runs 1 and 2. The sewage used in run 2 contained fewer sediments than the sewage used in run 1.

Clogging appeared to decrease the permeability of the formation in a non-uniform manner. A sharp decrease in permeabilities was observed near the well and the decrease was more gradual farther from the well. Relative permeability, which is defined as the ratio of permeability of the wetted zone after sewage injection to that of the same zone after tap water injection and after steady state conditions are attained, was found to be an excellent measure of the decrease in permeability caused by sewage injection. The relative permeabilities, which were computed for different times and distances from the well, are listed in Tables 3-8. Plots of these relative permeabilities versus $(\frac{r}{r_w})$ are shown in Figures 13-18.

A sharp decrease in permeability of the wetted zone during the first hour of injection was observed for all three plans. The decrease ranged from 10-20% for plan 1, 26-29% for plan 2, and 10-15% for plan 3. A similar decrease was also observed at the Riverhead Project in New York (6). This sharp decrease in permeability may be attributed to the degassification of the injected liquid caused by the presence of organic and biological impurities which cause a decrease in surface tension of the fluid. From data obtained at the Riverhead Project, there was some evidence that lowered surface tension in the injection fluid was associated with greater entrance head loss which affects permeability measurements directly. Higher initial concentrations cause greater decreases in permeability as shown in the results from plan 2A where the decrease was about 27% of that computed at the steady state, while for plans 1 and 3, where lower concentrations were injected, the decrease was only about 15%.

The rate of decrease in permeability is dependent on three factors: concentration of sediments in the injected fluid, duration of the injection process, and distance from the center of the injection well. At low concentrations, the decrease in permeability was slow but as concentrations

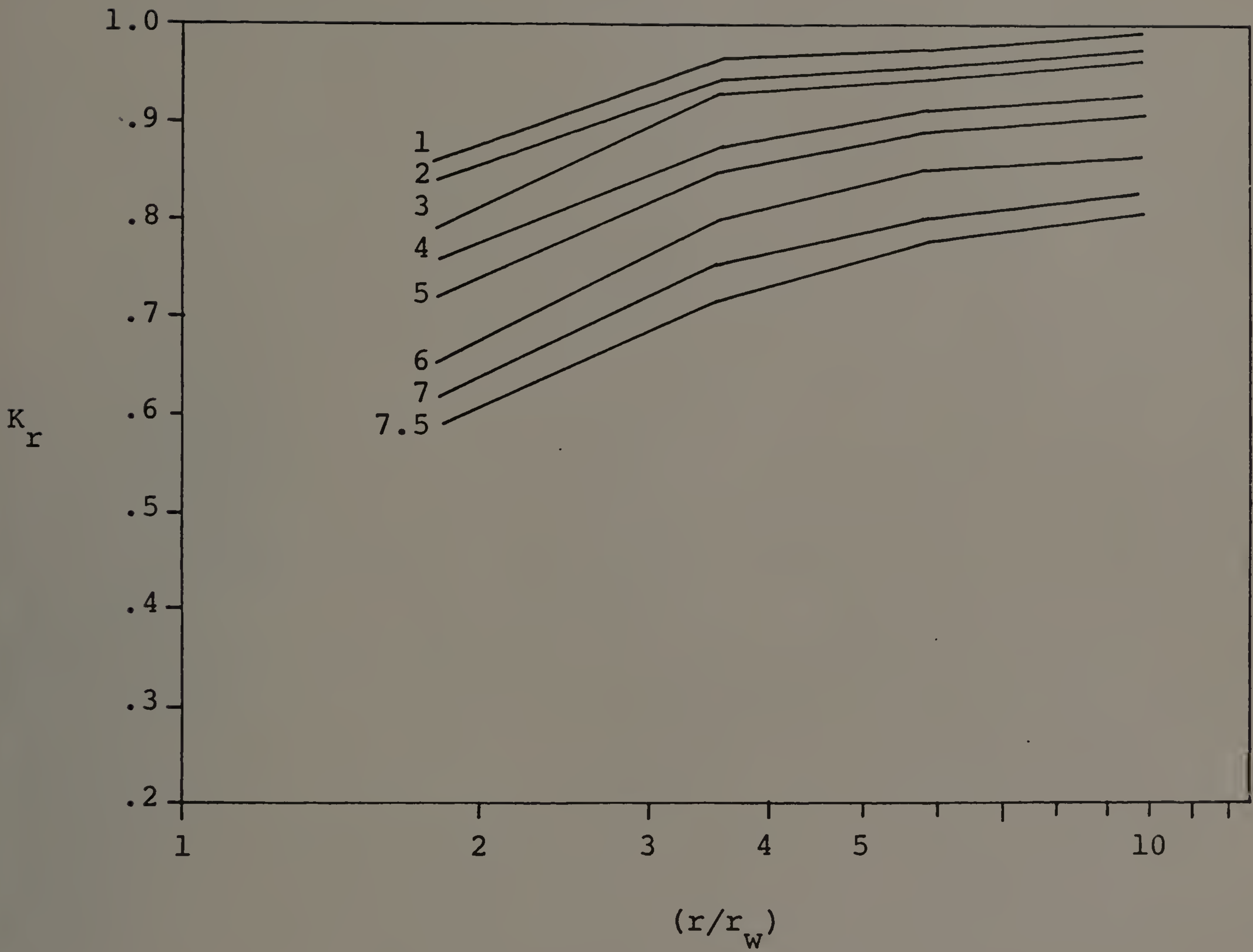


FIGURE 13

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 1, PLAN 1)

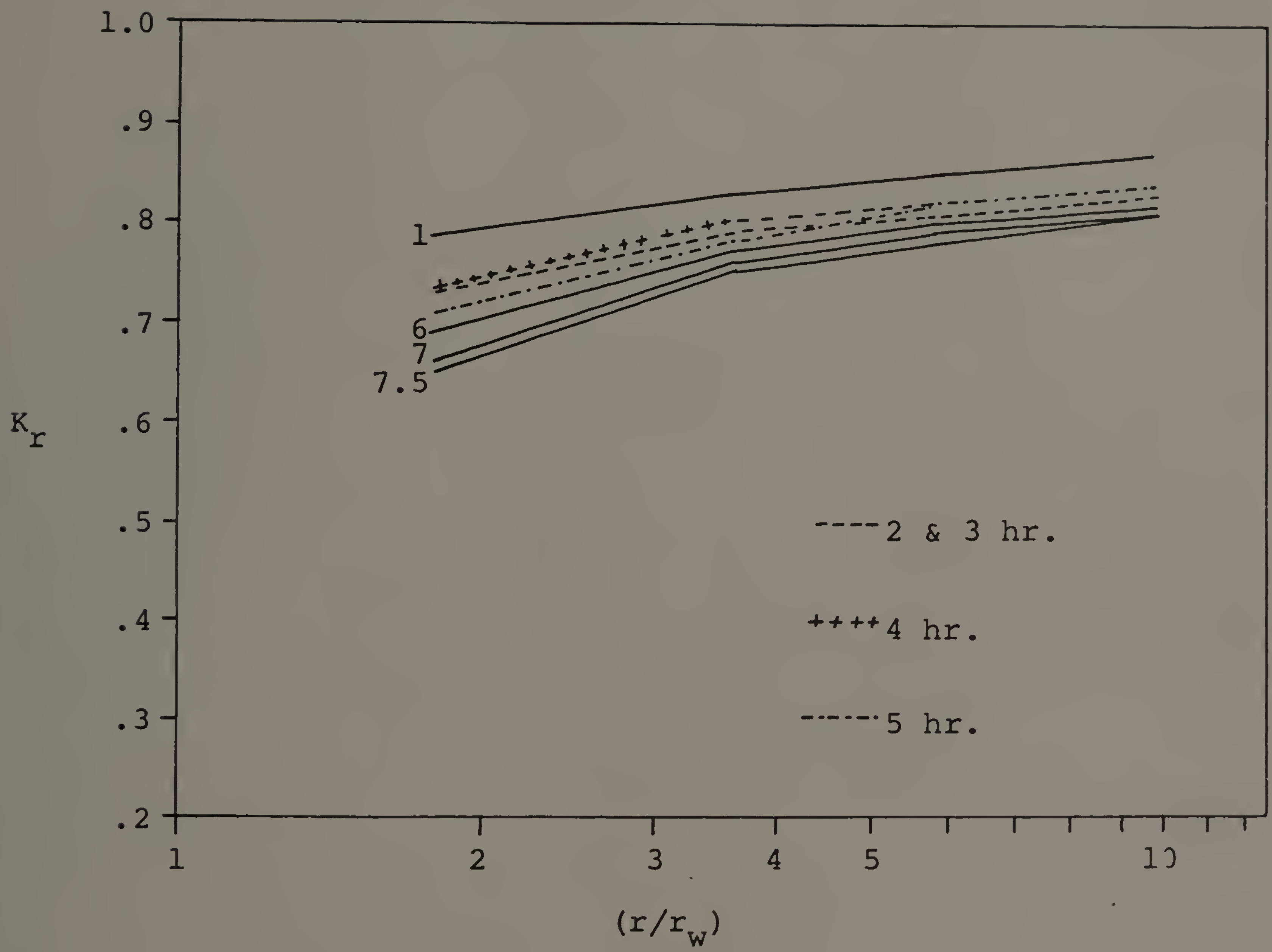


FIGURE 14

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 2, PLAN 1)

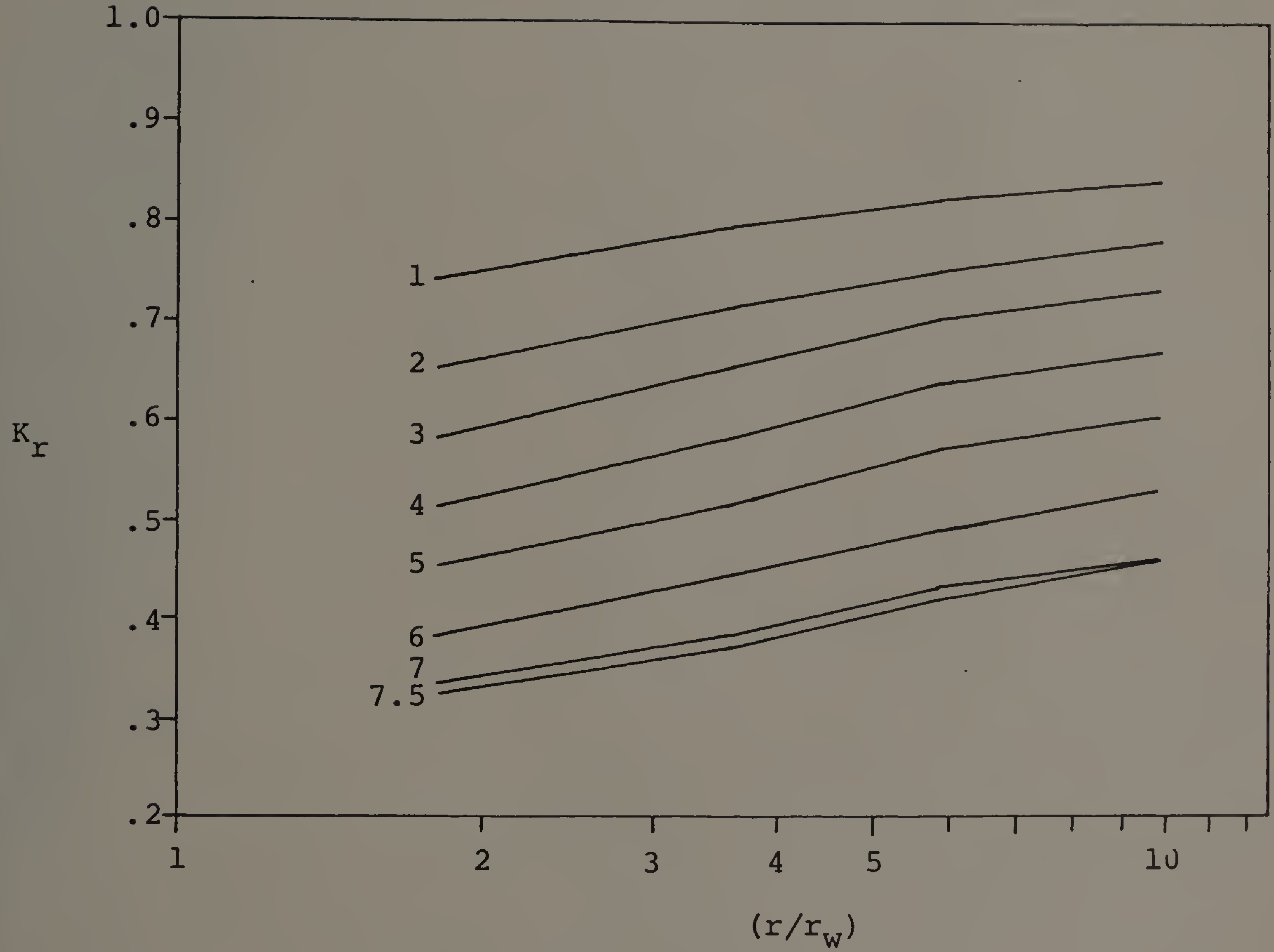


FIGURE 15

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 1, PLAN 2A)

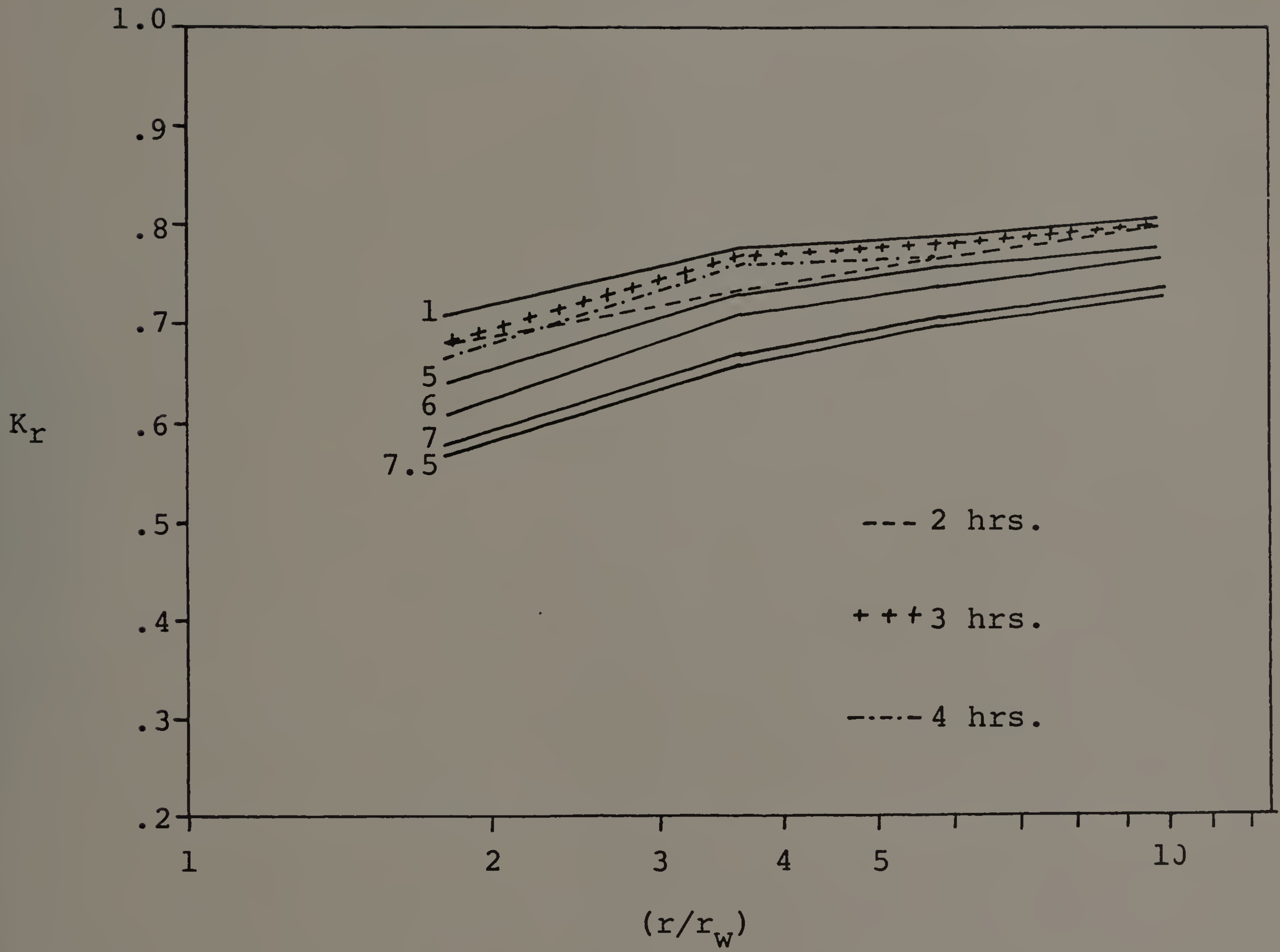


FIGURE 16

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 2, PLAN 2A)

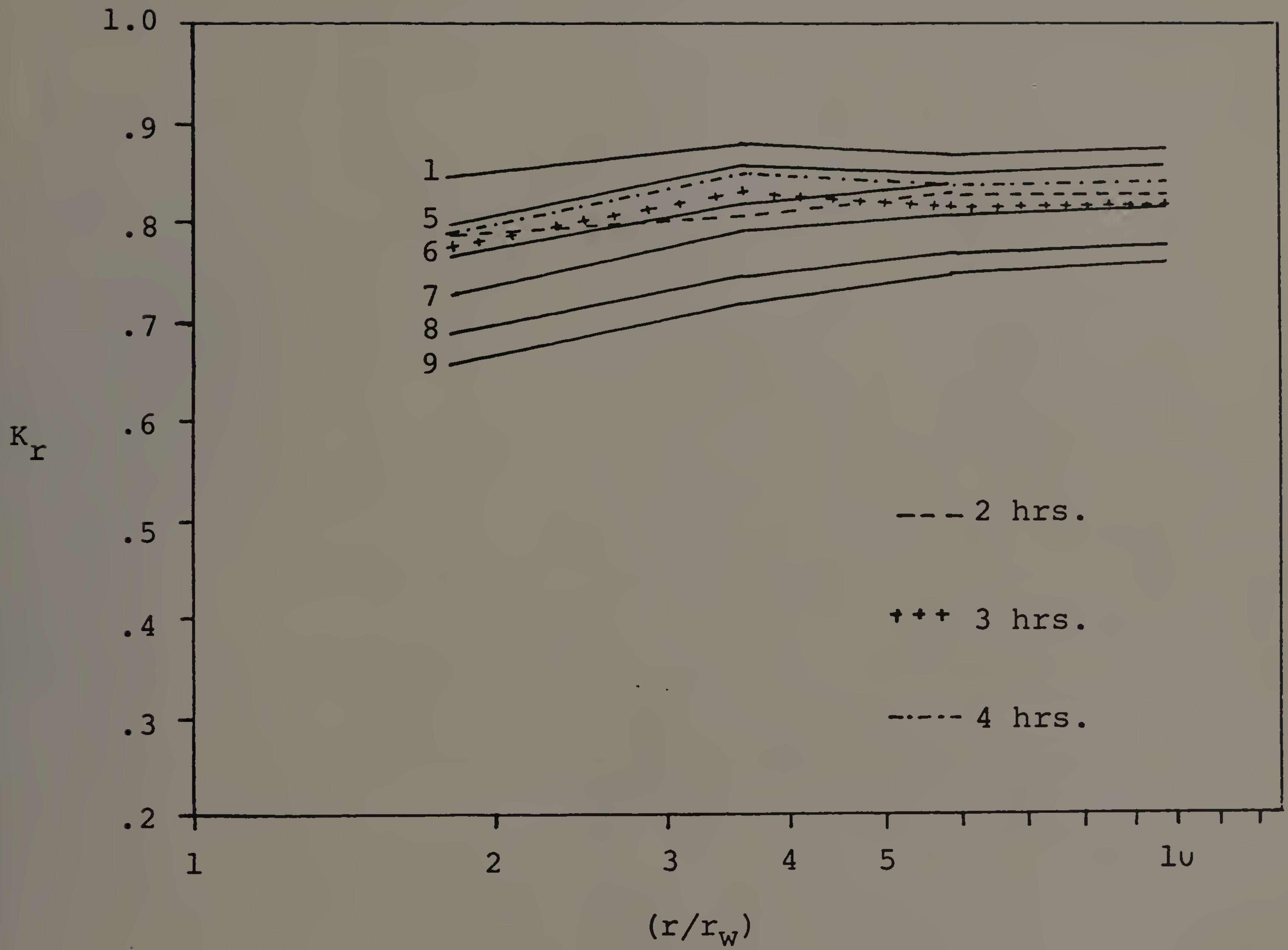


FIGURE 17

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 1, PLAN 3)

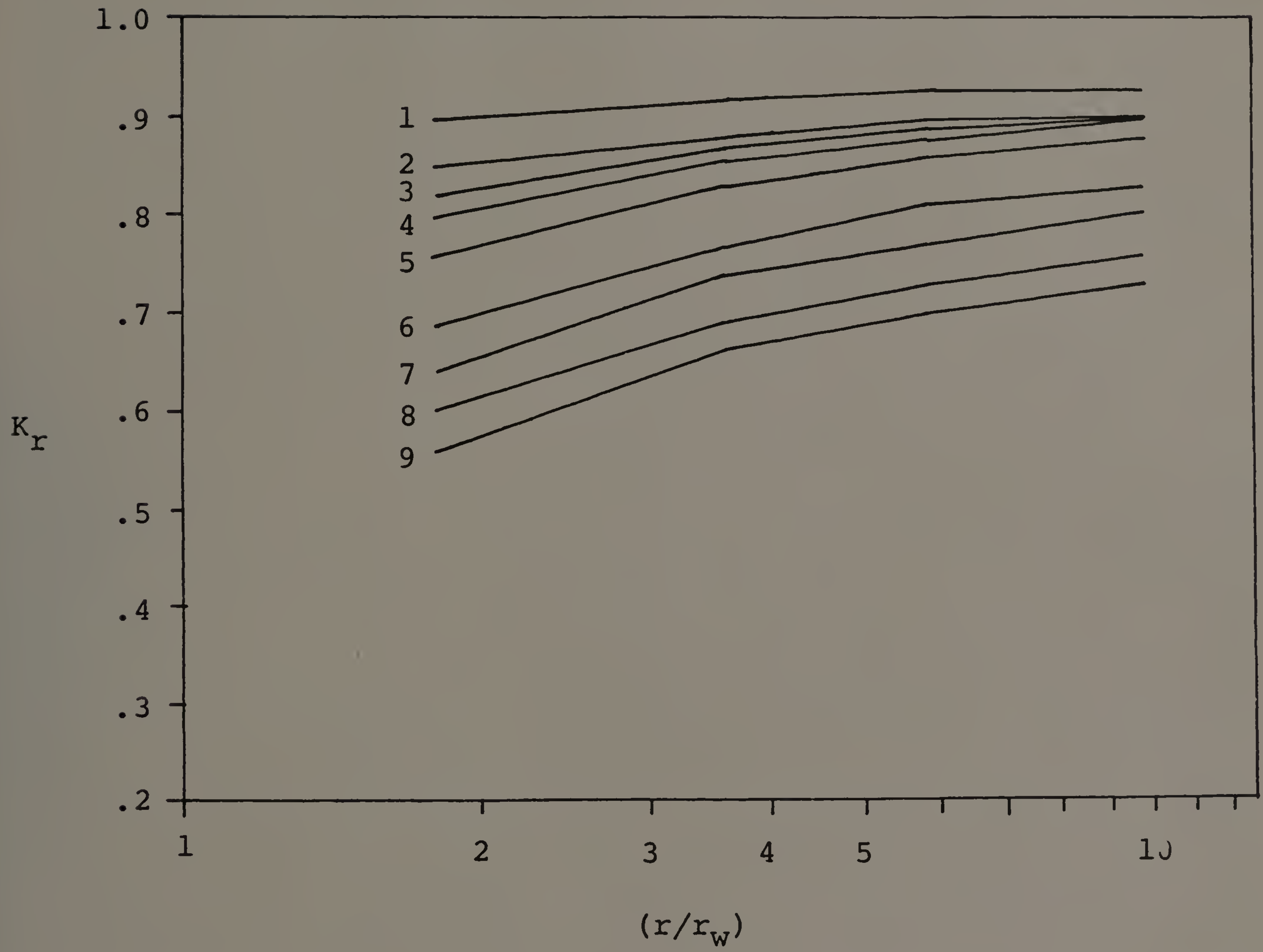


FIGURE 18

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 2, PLAN 3)

increased there was a sharp decrease in permeability for runs 1 and 2 following plan 1.

Plan 3 behaved in a different manner. During the initial stages, when concentrations were low, a small decrease in permeability occurred but as the concentration increased to 20% a sharp decrease occurred. At the end of the higher concentration period, the permeability was decreasing more slowly.

The gross decrease in permeability was always less in plan 1 than it was in plan 3, which was less than in plan 2A. Thus, the initial concentration of injected fluid was important in determining the magnitude of the gross decrease in permeability.

In an attempt to investigate the effect of shorter rainstorms on the injection process without changing the sand in the simulator, scheme 1 was carried out. The steady state piezometric surface obtained by injecting only tap water and the final piezometric surface at the end of the 3 consecutive runs following plan 2B are shown in Figure 19. Data obtained for piezometric surfaces at the end of every 30 minutes between the beginning and the end of each run is presented in Tables 13, 15, and 17 of Appendix B.

The slope of the piezometric surface near the well increased as sediments clogged the formation. The greatest head loss occurred in the well zone and in the first 2 feet of the formation. Pumping the system for 30 minutes using tap water was ineffective in recovering the head loss.

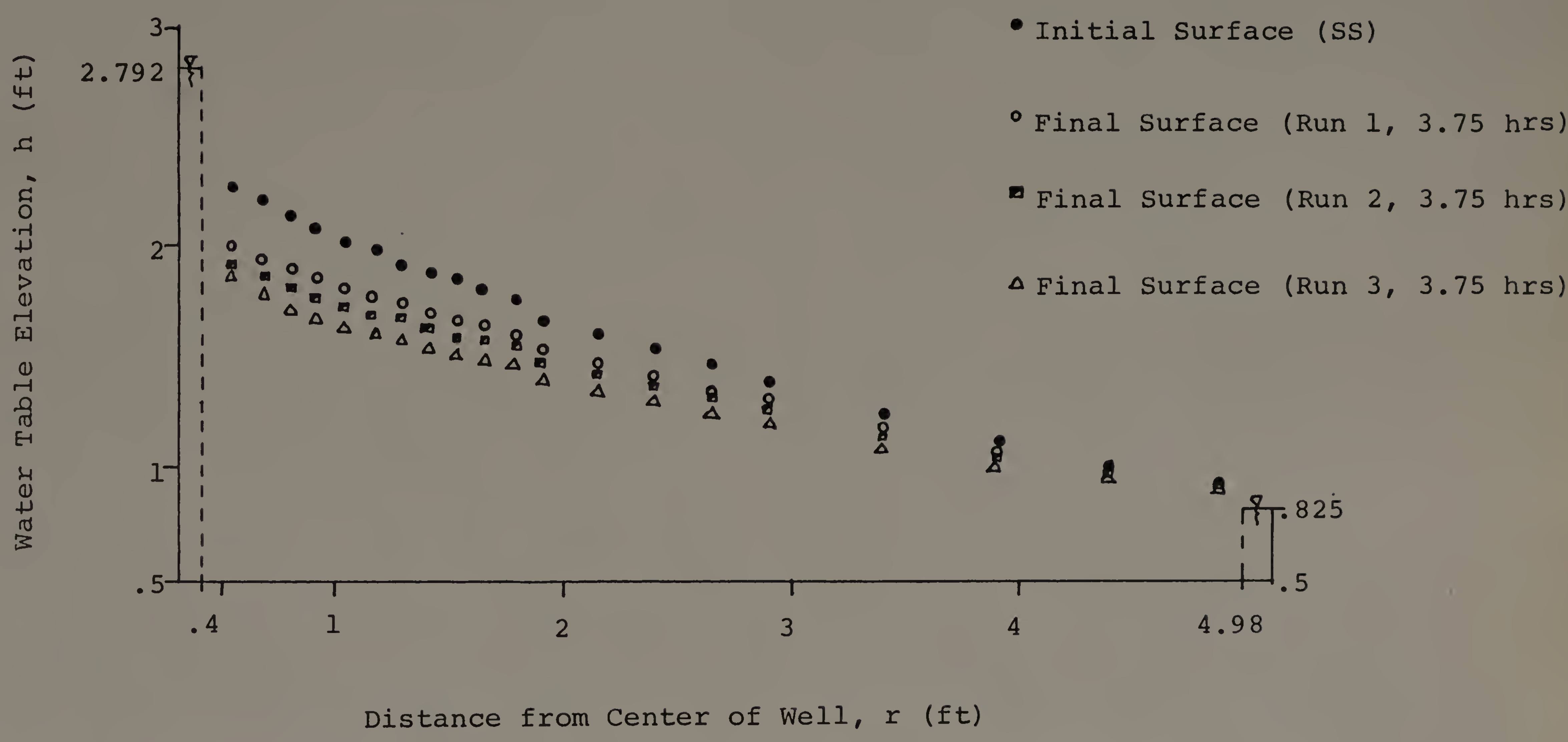


FIGURE 19

INJECTION PIEZOMETRIC SURFACES (RUNS 1, 2, and 3, PLAN 2B)

The rate of mass flux flow through the formation also decreased continuously as time proceeded. Tables 9-11 show the rate of flow Q in gallons per minute as computed for a radial system for each test run. Because of a short period of injection and successive pumping, the decrease in the rate of outflow was smaller than that observed during long periods of injection. At the end of run 1, Q was 86% of that measured at the steady state. Values of Q decreased to 76% and 63% at the end of runs 2 and 3 respectively.

When plan 2B was followed, permeability decreased slightly in run 1 with a sharp decrease of 32% near the well and only 16% decrease at a distance of 4 feet from the center of the well. Pumping helped in recovering some of the loss in permeability values by bringing it back to where it was at 90 minutes after injection for both the first and second runs. The final permeability decreased to about 50% of that computed at the steady state. Relative permeabilities, which were computed for different times and distances from the well, are listed in Tables 9-11. Plots of these relative permeabilities versus $(\frac{r}{r_w})$ are shown in Figures 20-22.

The final permeability and piezometric surface at the end of run 3, plan 2B were higher than they were at the end of run 1, plan 2A. This shows the effect of the length of the period of injection on both the piezometric surface and the permeability of the formation.

TIME HR.	Q GPM(CFS)
S S	7.724
.50	7.444
1.00	7.254
1.50	7.105
2.00	6.972
2.50	6.893
3.00	6.722
3.75	6.636

	RELATIVE PERMEABILITY AT				
H FT.	0.5 HR.	1.0 HR.	1.5 HR.	2.0 HR.	2.5 HR.
.719	.91	.86	.81	.77	.74
1.412	.93	.89	.85	.82	.80
2.341	.93	.89	.86	.84	.82
3.939	.96	.93	.91	.88	.88

(CONTINUED)

	REL. PERM. AT	
H FT.	3.0 HR.	3.75 HR.
.719	.70	.68
1.412	.76	.75
2.341	.79	.78
3.939	.85	.84

TABLE 9

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 1 PLAN 2F

TIME HR.	Q CFM(CFS)
S S	7.724
.50	6.985
1.00	6.714
1.50	6.482
2.00	6.441
2.50	6.249
3.00	6.135
3.75	5.928

	RELATIVE PERMEABILITY AT				
R FT.	0.5 HR.	1.0 HR.	1.5 HR.	2.0 HR.	2.5 HR.
.719	.78	.72	.68	.66	.62
1.412	.83	.78	.74	.73	.69
2.341	.85	.81	.77	.76	.73
3.939	.90	.86	.83	.82	.79

(CONTINUED)

	REL. PERM. AT	
R FT.	3.0 HR.	3.75 HR.
.719	.60	.57
1.412	.68	.64
2.341	.72	.69
3.939	.78	.76

TABLE 10

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT THE END OF EACH HOUR OF RUN 2 PLAN 2B

TIME HR.	Q CFM(CFS)
S S	7.724
.50	6.311
1.00	5.861
1.50	5.643
2.00	5.446
2.50	5.300
3.00	5.103
3.75	4.935

	RELATIVE PERMEABILITY AT						
R FT.	0.5 HR.	1.0 HR.	1.5 HR.	2.0 HR.	2.5 HR.	3.0 HR.	3.75 HR.
.719	.67	.60	.56	.52	.50	.47	.44
1.412	.71	.65	.61	.58	.57	.53	.50
2.341	.74	.68	.65	.62	.60	.58	.55
3.939	.81	.75	.72	.69	.67	.65	.62

(CONTINUED)

TABLE 11

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 3 PLAN 2B

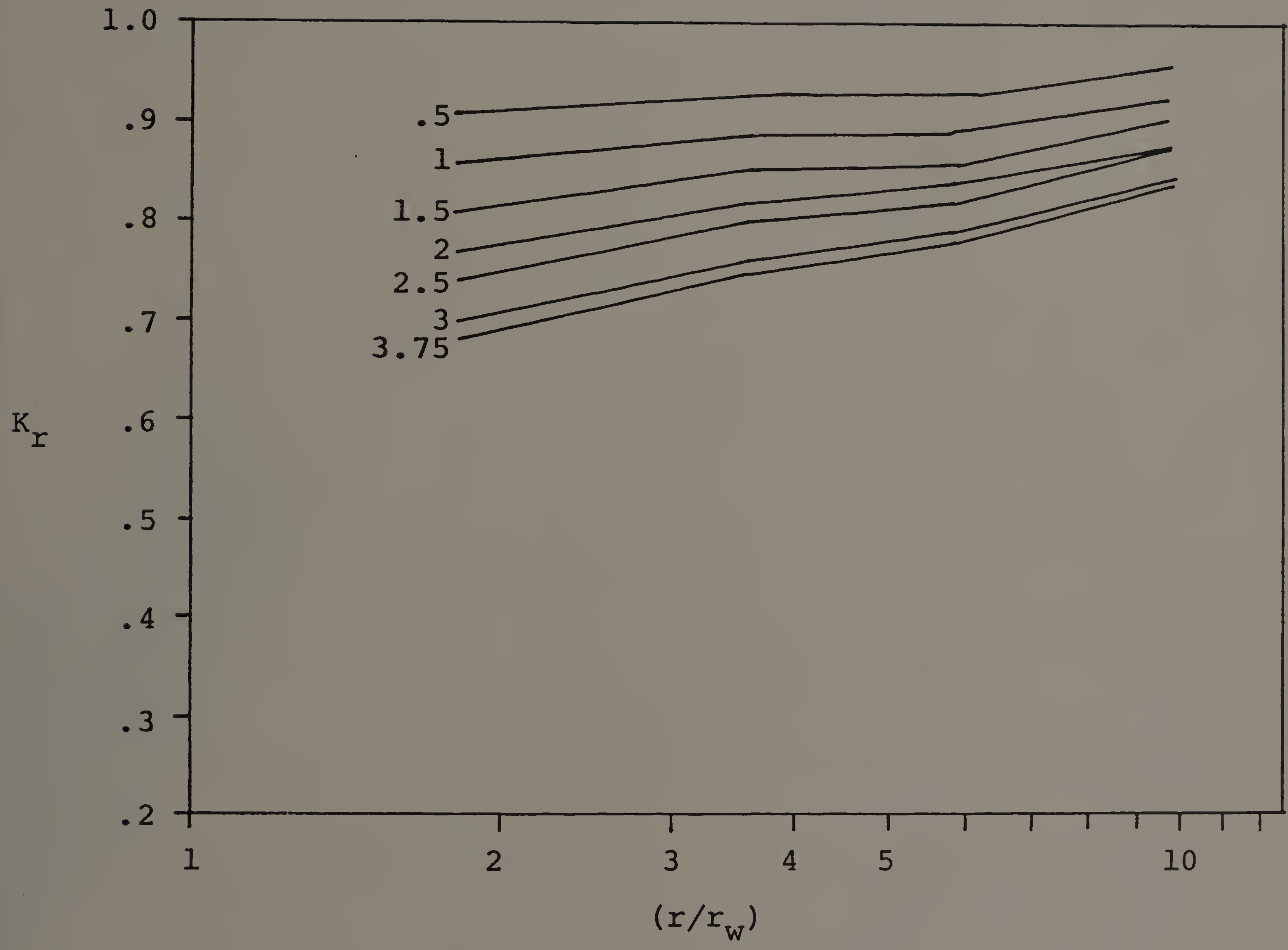


FIGURE 20

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 1, PLAN 2B)

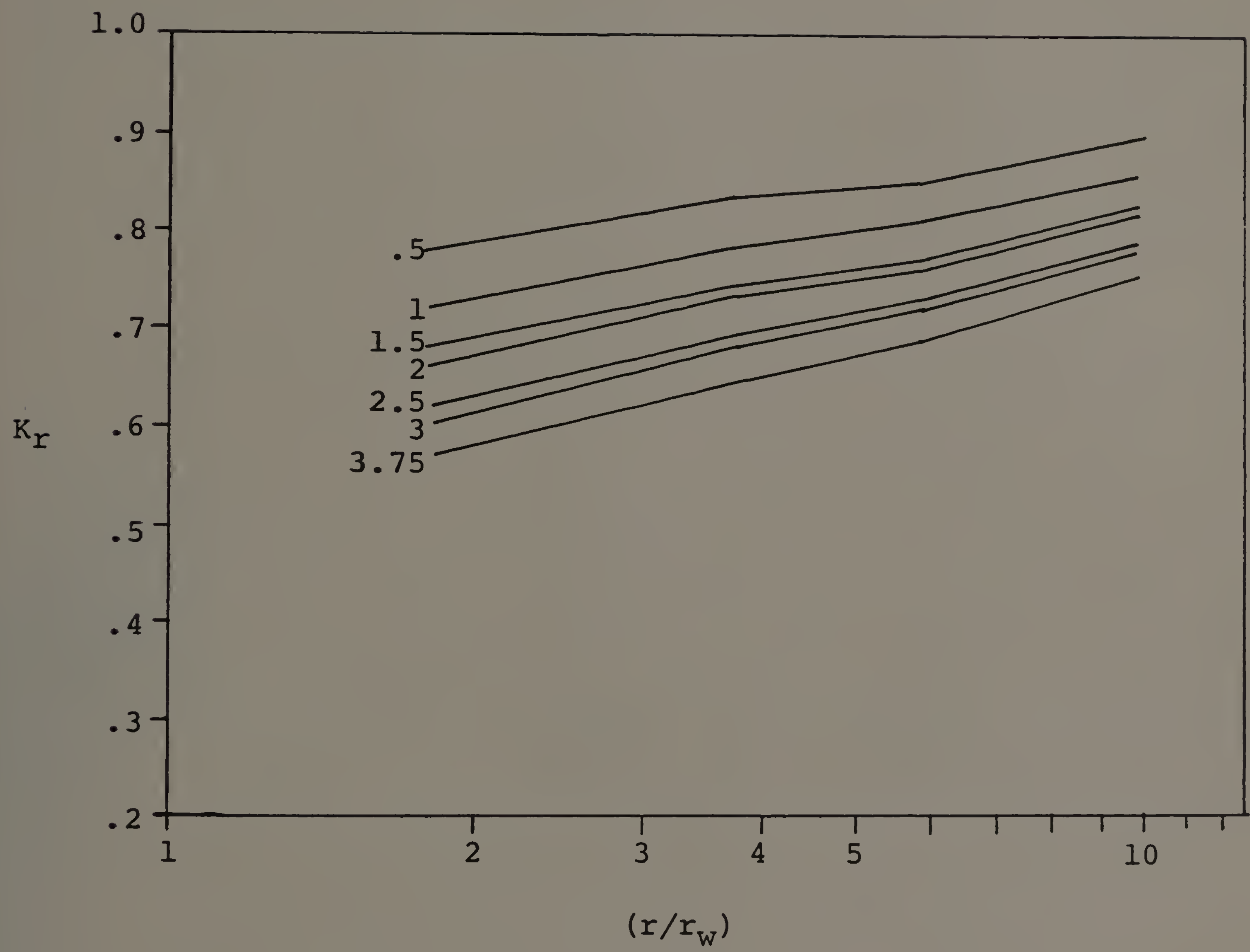


FIGURE 21

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 2, PLAN 2B)

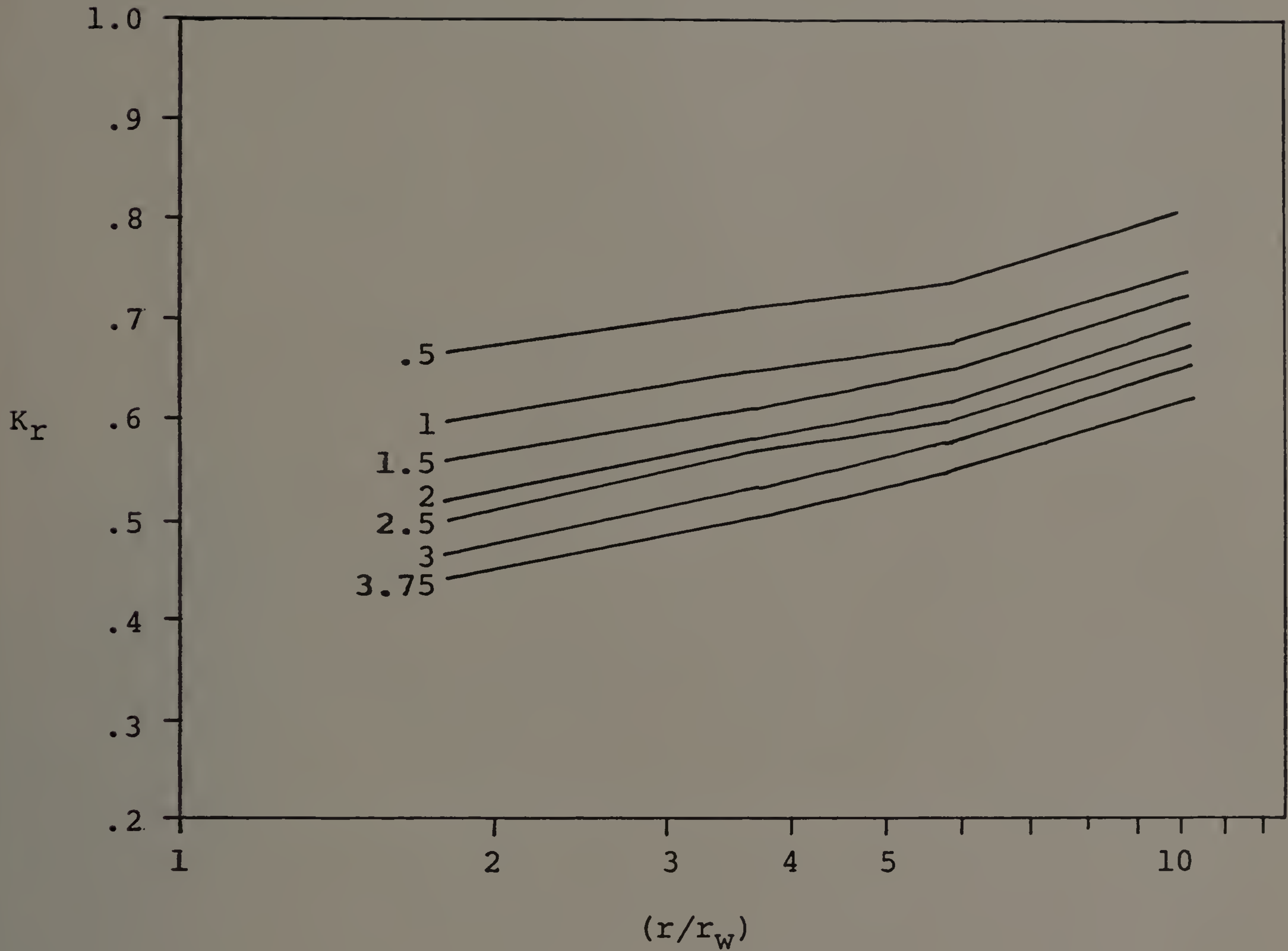


FIGURE 22

RELATIVE PERMEABILITY VS (r/r_w) WITH TIME (hr) AS THE PARAMETER (RUN 3, PLAN 2B)

The ineffectiveness of pumping in restoring the permeability of the formation suggests a need for a chemical or mechanical treatment to redevelop the formation for successive injections.

Scheme 2 was planned to investigate the effect of periodic pumping (15 minutes every 3 hours) over an extended period of injection without changing the sand in the simulator.

After recording the initial steady state piezometric surface (Figure 23), runs 1, 2, and 3 were carried out and manometer readings recorded at the end of each hour. Experimental data are presented in Tables 19, 21, and 23 of Appendix B.

Obviously some of the sediments which settled in the formation cannot be removed by subsequent pumping. The amount of sediments trapped in the formation increased as injection continued. This caused a continuous drop in the piezometric surface and a continuous decrease in the rate of outflow as shown in Tables 12-14. The rate of outflow decreased by 10% after 3 hours of injection and thus, due to pumping for 15 minutes, it had decreased only by 1% after 4 hours, and then decreased to 18% after 6 hours. Pumping at 6 hours recovered some of this loss, where the decrease was only 17% at the end of the first run. A similar decrease in the rate of outflow was measured for subsequent runs 2 and 3. After 3 hours, the decrease in Q for run 2 was 31% and for run 3 it was 46% of its value recorded at steady state. Pumping at the end of 3 hours recovered 4% of this loss for run 2 and 2% for run 3

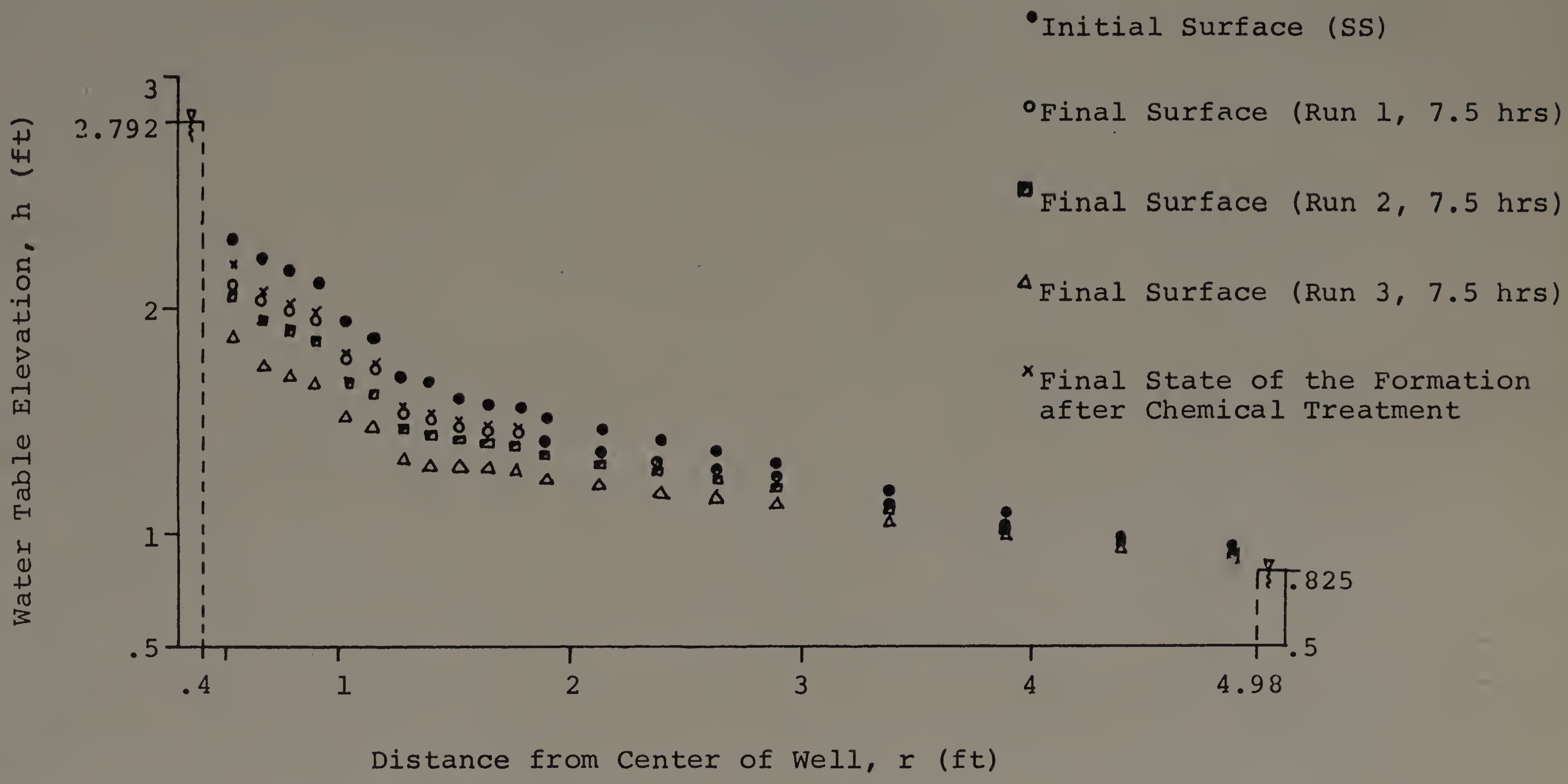


FIGURE 23

INJECTION PIEZOMETRIC SURFACES (RUNS 1, 2, and 3, PLAN 2A WITH PERIODIC PUMPING)

TIME HR.	Q CFM(CFS)
S S	6.200
1.0	5.922
2.0	5.715
3.0	5.589
4.0	5.515
5.0	5.197
6.0	5.067
7.0	5.332
7.5	5.150

	RELATIVE PERMEABILITY AT				
H FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.92	.85	.80	.80	.71
1.412	.94	.89	.87	.84	.78
2.341	.95	.90	.88	.87	.80
3.939	.96	.91	.89	.88	.83

(CONTINUED)

	RELATIVE PERMEABILITY AT		
H FT.	6 HR.	7 HR.	7.5 HR.
.719	.51	.74	.69
1.412	.71	.80	.76
2.341	.78	.83	.79
3.939	.80	.85	.82

TABLE 12

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 1 PLAN 2A (SCHEME 2)

TIME HR.	Q GPM(CFS)
S S	6.200
1.0	4.814
2.0	4.524
3.0	4.255
4.0	4.489
5.0	4.276
6.0	4.090
7.0	4.508
7.5	4.326

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.69	.61	.56	.60	.55
1.412	.74	.68	.63	.67	.63
2.341	.76	.71	.67	.71	.67
3.939	.78	.74	.70	.74	.70

(CONTINUED)

	RELATIVE PERMEABILITY AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.719	.51	.59	.56
1.412	.59	.67	.64
2.341	.63	.70	.67
3.939	.66	.73	.70

TABLE 13

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT THE END OF EACH HOUR OF RUN 2 PLAN 2A (SCHEME 2)

TIME HR.	Q GPM(CFS)
S S	6.200
1.0	3.896
2.0	3.648
3.0	3.361
4.0	3.490
5.0	3.276
6.0	3.135
7.0	3.583
7.5	3.381
F S	4.884

	RELATIVE PERMEABILITY AT				
R FT.	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.719	.48	.43	.39	.40	.36
1.412	.57	.52	.48	.49	.45
2.341	.60	.56	.52	.53	.49
3.939	.64	.60	.55	.56	.53

(CONTINUED)

	RELATIVE PERMEABILITY AT			
R FT.	6 HR.	7 HR.	7.5 HR.	F S
.719	.34	.42	.38	.70
1.412	.43	.51	.47	.75
2.341	.46	.55	.51	.76
3.939	.50	.59	.55	.80

TABLE 14

THE RATE OF OUTFLOW AND RELATIVE PERMEABILITY AT

THE END OF EACH HOUR OF RUN 3 PLAN 2A (SCHEME 2) when measured at the end of 4 hours total injection time. The loss in Q was 34% for run 2 and 49% for run 3 at the end of 6 hours and because of pumping at this time, the final loss in Q, at a total elapsed time of 7.5 hours, was 30% for run 2 and 46% for run 3. This indicates that periodic pumping slows down the rate at which Q decreases.

Prior to starting run 2, and having rested overnight, the system was pumped for 30 minutes. It would be expected that the flow rate would be more than 83% of the steady state value, however, it was 77% which suggests that the sediments which were not driven out by pumping were bio-chemical in nature. Sediments left in the formation for 8-9 hours enhance microbial growth. Again this supports the need for chemical treatment to help reduce microbial growth.

At the end of the second run, and after pumping for 15 minutes, 22 gallons of tap water containing 10 ppm sodium hypochlorite was injected into the formation. At this concentration there was little effect in reducing microbial growth. In fact, after one hour of injection, an additional 6% loss in flow rate was recorded. This suggested the need for injecting a higher concentration of sodium hypochlorite. Therefore, 22 gallons of tap water containing 250 ppm sodium hypochlorite were injected into the formation and the formation was then left overnight. It was then injected with tap water until a steady state was reached where the flow rate was 79% of its original value. In other words there was a

25% recovery in flow rate compared to the value recorded at the end of run 3 (54%). The piezometric surface at this final state was higher than the final surface at the end of run 1 as shown in Figure 23. The recovery in permeability was 25%. These data suggest that chemical treatment reduces biological growth and helps in recovering original formation characteristics.

Frequent pumping was not effective in recovering permeability loss as shown in Tables 12-14. The permeability decreased continuously and the greatest decrease was near the well. Final permeabilities at the end of each run and the permeability recorded after the injection of 250 ppm sodium hypochlorite are shown in Figure 24.

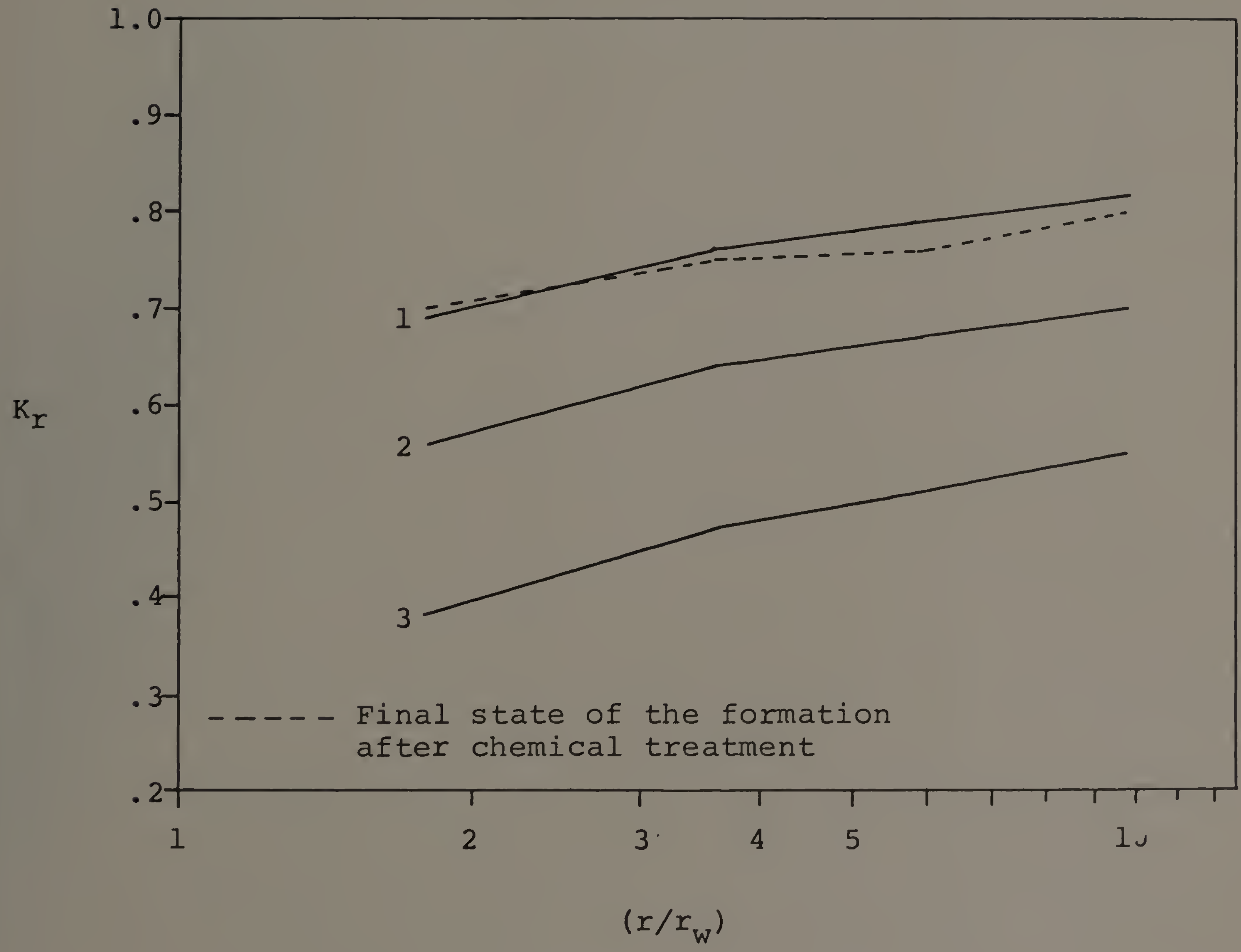


FIGURE 24

RELATIVE PERMEABILITY VS (r/r_w) AT THE END OF RUNS 1, 2, AND 3, SCHEME 2

SUMMARY AND CONCLUSIONS

Studies were conducted to investigate the change in physical and hydraulic properties of soils due to movement of water containing sediments into and through a soil formation. They involved understanding how changes in rate of injection were affected; the nature and mechanics of clogging; and such hydraulic properties as piezometric surface and permeability.

From the data obtained the following conclusions are drawn.

1) There is a boundary entrance head loss at the well-formation interface. This is evident because the sudden drop in the experimental piezometric surface does not conform with the Dupuit injection surface. This suggests that field permeability calculations, based upon pumping tests alone, are erroneous when a Dupuit surface is assumed.
2) The decrease in piezometric surface level, rate of outflow, and permeability are functions of three main variables: concentration of the injected fluid; duration of injection; and distance from the injection well.
3) The greater the initial concentration of the injected fluid, the greater the decrease in both piezometric surface and transmissibility of the porous material.

4) Successive short periods of injection, followed by pumping, result in less change in physical and hydraulic properties than a single long injection period. For example, 3 successive 3.75 hour periods of injection, followed by 30 minutes of pumping resulted in less change in these properties than one 7.5 hour injection period.
5) Pumping at the end of the injection period is an effective procedure in backwashing some of the sediments which have clogged the formation, specifically for short injection periods. For longer periods, pumping was not effective, thus indicating the need for a chemical or mechanical treatment. Periodic pumping during injection is more effective than pumping at the end of injection, but not if frequent successive injections are desired.
6) Most of the sediments which are not backwashed are biochemical in nature and chemical treatment is more successful in redeveloping the transmission properties of the formation. The injection of water containing 250 ppm sodium hypochlorite at the end of a sewage injection period was found to be the most effective method in reducing microbial growth and chemical degradation.

Future studies should emphasize the following:

1) Investigate the physics and mathematics of boundary head loss and the factors that contribute to it, such as surface tension, capillarity, and loss of momentum.

2) Examine the nature and mechanisms of clogging and the clogability of different kinds of sewage and its effect on the physical properties of porous materials.
3) Estimate the portion of outflow that is due to the unsaturated zone and, therefore, make more exact estimates of changes in permeability.
4) Examine post-injection treatments that most efficiently backwash sediments and restore the formation to its original state.
5) Investigate the effects of injecting overflows on the chemical and biological properties of soils.

LITERATURE CITED

1. Adrian, D. D. Class Notes, C.E. 775. Environmental Systems Simulation.

2. Allison, L. E. 1947. Effect of Microorganisms on Permeability of Soil under Prolonged Submergence. Soil Sci. 63:439-450.

3. Amramy, A. 1964. Waste Treatment for Ground Water Recharge. Proceedings of the Second Underground Water Pollution Control Conference, Tokyo, Japan. 147-168.

4. Avnimelech, Y., and Z. Nevo. 1964. Biological clogging of sand. Soil Sci. 98:222-226.

5. Babbitt, N. E., and D. N. Caldwell. 1969. The free surface around and interference between gravity wells. University of Ill. Eng. Expt. Station. Bulletin series No. 374.

6. Baffa, J. J. 1970. Injection well experience at Riverhead, N.Y. Journal American Water Works Association. 66:41.

7. Baffa, J. J., and N. J. Bartilucci. 1967. Wastewater reclamation by ground water recharge on Long Island. Journal Water Pollution Control Federation. 39:431.

8. Bliss, E. S., and C. E. Johnson. 1952. Some factors involved in ground water replenishment. Trans. Am. Geophys. Union. 33:547-558.

9. Butler, R. S., and G. T. Orlob. 1954. Underground Movement of Bacterial and Chemical Pollution. Journal American Water Works Association. 46:95.

10. California State Water Quality Control Board. 1954. Studies of wastewater reclamation and utilization. Pub. No. 9.

11. Duguid, J. P., and J. F. Wilkinson. 1953. The influence of cultural conditions on polysaccharides production by Aerobacter aerogenes. Jour. Gen. Microbiol. 9:174-189.

12. Jones, J. H., and G. S. Taylor. 1965. Septic tank effluent percolation through sands under laboratory conditions. Soil Sci. 99:301-309.

13. Laverty, F. B.; R. Stone; and L. A. Meyerson. 1961. Reclaiming Hyperion Effluent. Proceedings of American Society of Civil Engineers. SA-6.

14. Lepri, R. Environmental Engineering Program, Dept. of Civil Engineering, Univ. of Mass. (private communication.)

15. Lynch, D. L. 1960. The effect of Rose bengal on polysaccharide production in soils. Can. Jour. Microbiol. 6:673-674.

16. Macalla, T. M. 1944. Factors affecting the percolation of water through a layer of Loessial soil. Soil Sci. Soc. Am. Proc. 9:12-16.

17. Martin, J. P. 1944. Microorganisms and soil aggregation: I. origin and nature of some aggregating substances. Soil Sci. 59:163-174.

18. Martin, J. P. 1946. Microorganisms and soil aggregation: II. Influence of Bacterial Polysaccharides on Soil Structure. Soil Sci. 61:157-166.

19. McGauhey, P. H. 1968. Engineering Management of Water Quality. 1st Edition. McGraw-Hill Book Co., New York.

20. Mitchell, R., and Z. Nevo. 1964. Effect of bacterial polysaccharide accumulation on infiltration of water through sand. Appl. Microbiol. 12:219-223.

21. Sanitary Eng. Res. Lab., University of California. 1955. Studies in Waste Reclamation. Tech. Bul. 13. I.E.R. Series 37.

22. Schief, L., and C. E. Johnson. 1958. Some methods in alleviating surface clogging in water spread with emphasis on filters. Trans. Am. Geophys. Union. 39:292-297.

23. Skodje, M. T. 1969. The use of gravity shafts for ground water recharge. Journal American Water Works Association. 61:245.

24. Sniegocki, T. R. 1963-1965. Artificial Recharge in the Grand Prairie Region, Arkansas. Geological Survey, Water Supply Papers, 1615, A through G, U.S. Government Printing Office.

25. Task Group Report. 1963. Design and Operation of Recharge Basins. Journal American Water Works Association. 55:697.

26. Task Group Report. 1965. Experience with injection wells for artificial ground water recharge. Journal American Water Works Association. 61:246.

27. Thomas, R. C.; W. A. Schwartz; and T. W. Bendixen. 1966. Soil chemical changes and infiltration rate reduction under sewage spreading. Soil Sci. Soc. Am. Proc. 30:641-646.

28. Thomas, R. E.; W. A. Schwartz; and T. W. Bendixen. 1968. Pore gas composition under sewage spreading. Soil Sci. Soc. Am. Proc. 32:419-423.

29. Wesner, G. M., and D. C. Baier. 1970. Injection of Reclaimed Wastewater into Confined Aquifers. Journal American Water Works Association. 62:203.

30. Winneberger, J. H.; L. Francis; S. A. Klein; and P. H. McGauhey. 1960. Biological aspects of failure of septic tank percolation system. Final Rep. School of Public Health, University of California, Berkeley.

APPENDIX A

THE LEAST SQUARED DEVIATIONS METHOD OF CALCULATING UNCONFINED FLOW PARAMETERS

The minimum sum of the squared deviations method was utilized to find the best fit between the theoretical and experimental piezometric surfaces. The procedure is as follows (1):

Let h_{t_i} = The theoretical piezometric height at a certain time and a certain distance from the center of the well.

h_{m_i} = The experimental piezometric height at a certain time and a certain distance from the center of the well.

and $\phi = \frac{Q}{\pi K}$

Equation 4 can be written as:

$$h_{t_i}{}^2 = h_w{}^2 - \phi \ln \left(\frac{r_i}{r_w}\right)$$

or

$$h_{t_i} = \left(h_w{}^2 - \phi \ln \left(\frac{r_i}{r_w}\right)\right)^{1/2}$$

Defining σ such that

$$\sigma = \sum_{i=1}^{i=n} (h_{m_i} - h_{t_i})^2 = \sum_{i=1}^{i=n} \left(h_{m_i} - \left(h_w{}^2 - \phi \ln \left(\frac{r_i}{r_w}\right)\right)^{1/2}\right)^2$$

and noting that the minimum value of σ is obtained when

$$\frac{d\sigma}{d\phi} = 0 = \frac{d}{d\phi} \sum_{i=1}^{i=n} \left(h_{m_i} - \left(h_w{}^2 - \phi \ln \left(\frac{r_i}{r_w}\right)\right)^{1/2}\right)^2$$

which yields after differentiating and rearranging:

$$\sum_{i=1}^{i=n} \left(\frac{h_{m_i}}{\sqrt{h_w{}^2 - \phi \ln \left(\frac{r_i}{r_w}\right)}} - 1\right) = 0 \qquad \text{(A-1)}$$

where n represents the number of manometer readings recorded.

To solve for ϕ at different distances from the well, the formation was divided into 4 sections as follows: Section 1 included the distance from the well interface to 1.041 feet, which gives an average distance of .719 feet and consisted of 5 manometers at .547, .670, .794, .918, and 1.041 feet. Section 2 included the distance from 1.041 feet to 1.783 feet with an average distance of 1.412 feet. It contained 6 manometers at 1.165, 1.289, 1.412, 1.536, 1.660, and 1.783 feet. Section 3 included the distance from 1.783 feet to 2.899 feet, with an average distance of 2.341 feet and contained 5 manometers at 1.907, 2.153, 2.401, 2.650, and 2.899 feet. Section 4 included the distance from 2.899 feet to the outlet chamber, 4.980 feet, with an average distance of 3.939 feet. It contained 4 manometers at 3.393, 3.888, 4.383, and 4.878 feet.

According to these sections, $n_1=5$, $n_2=6$, $n_3=5$, and $n_4=4$. Equation A-1 represents ϕ implicitly in the number of summation steps n_i. An economical way to solve for ϕ is the use of the gradient method, which consists of one dimensional regression analysis based on finding the minimum of the sum of squared deviations by using a finite iteration procedure and is shown in Table A-1.

When ϕ is obtained for a certain time and since the rate of flow "Q" and temperature correction "TC" are known for that time, the standard values of permeability can be easily obtained. Permeability values obtained for each run are presented in Tables 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, and 24 of Appendix B.

```
PROGRAM P
DIMENSION H(20),R(20)
READ,(R(I),I=1,20)
READ,(H(I),I=1,20)
READ,Q,TC,C1
M1=1     $M2=5      $ K=1
C=7795.
5000 N1=1000
N2=25000
INC=1000
M=1
500 DO 200 IFI=N1,N2,INC
FI=IFI
FUN=0.0
PRINT ,FUN
DO 100 I=M1,M2
S=C-FI*R(I)
IF(S .LT. 0.0)GO TO 10
FUN=FUN+R(I)*((H(I)/(SQRT((C-FI*R(I))/1000.)))-1.)
100 CONTINUE
IF(FUN .GT. 0.0) GO TO 300
TEMP=FUN    $ IF=IFI
200 CONTINUE
300 GO TO (10,20,30,40),M
10 N1=IF    $ N2=N1+300    $ INC=100
M=2    $ GO TO 500
20 N1=IF    $ N2=N1+100    $INC=10
M=3    $ GO TO 500
30 N1=IF     $ N2=N1+10    $ INC=1
M=4     $ GO TO 500
40 CONTINUE
A=IF
KK=C1*Q*TC*1./A
PRINT 600,KK
600 FORMAT(3X,*VALUE OF PERMEABILITY=*,F10.4,*GAL/DAY/SQ.FT.*,/)
GO TO (11,12,13,14),K
11 M1=6  $ M2=11   $ K=K+1    $ GO TO 5000
12 M1=12  $ M2=16   $ K=K+1   $ GO TO 5000
13 M1=17  $ M2=20   $ K=K+1   $ GO TO 5000
14 STOP
END
ENDPROG
DATA
```

TABLE A-1

GRADIENT SOLUTION OF EQUATION A-1

APPENDIX B

EXPERIMENTAL DATA OF PIEZOMETRIC SURFACE AND PERMEABILITY AS FUNCTIONS OF TIME AND POSITION

TABLE B-1

RUN 1 PLAN 1

R FT.	PIEZOMETRIC SURFACE IN FEET AT					
	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.316	2.183	2.150	2.116	2.083	2.054
.670	2.191	2.066	2.041	2.016	1.979	1.950
.794	2.083	1.974	1.950	1.925	1.891	1.867
.918	1.991	1.891	1.871	1.850	1.817	1.729
1.041	1.908	1.817	1.792	1.775	1.745	1.725
1.165	1.833	1.750	1.733	1.716	1.687	1.666
1.289	1.758	1.687	1.666	1.645	1.629	1.612
1.412	1.699	1.633	1.616	1.599	1.583	1.566
1.536	1.633	1.579	1.566	1.550	1.533	1.516
1.660	1.591	1.541	1.525	1.516	1.495	1.483
1.783	1.554	1.504	1.491	1.479	1.462	1.450
1.907	1.491	1.441	1.433	1.420	1.408	1.395
2.153	1.416	1.375	1.366	1.358	1.341	1.333
2.401	1.350	1.316	1.308	1.299	1.291	1.279
2.650	1.275	1.250	1.241	1.233	1.225	1.216
2.899	1.191	1.175	1.116	1.150	1.150	1.145
3.393	1.050	1.041	1.033	1.033	1.029	1.020
3.888	.966	.958	.954	.950	.950	.945
4.383	.900	.895	.895	.895	.895	.891
4.878	.858	.858	.858	.858	.858	.854
4.980	.825	.825	.825	.825	.825	.825

TABLE B-1 (CONTINUED)

	PIEZOMETRIC SURFACE FT. AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.397	2.792	2.792	2.792
.547	1.999	1.933	1.904
.670	1.900	1.846	1.808
.794	1.817	1.766	1.737
.918	1.750	1.704	1.670
1.041	1.683	1.641	1.616
1.165	1.629	1.591	1.566
1.289	1.575	1.541	1.516
1.412	1.533	1.500	1.479
1.536	1.483	1.458	1.437
1.660	1.454	1.425	1.408
1.783	1.420	1.395	1.379
1.907	1.366	1.341	1.333
2.153	1.308	1.287	1.275
2.401	1.258	1.241	1.233
2.650	1.199	1.183	1.175
2.899	1.133	1.116	1.108
3.393	1.016	1.008	.999
3.888	.941	.933	.933
4.383	.891	.887	.883
4.878	.850	.850	.850
4.930	.825	.825	.825

TABLE B-2

RUN 1 PLAN 1

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	773.952	695.026	669.788	649.768	610.665
1.412	964.977	925.754	910.973	900.203	843.552
2.341	1101.663	1081.798	1055.379	1043.329	1003.635
3.939	1250.823	1243.832	1224.023	1209.551	1166.759

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT 5 HR.	6 HR.	7 HR.	7.5 HR.
585.695	554.768	500.115	481.528
817.201	772.019	720.810	700.270
977.078	932.114	877.453	858.298
1139.589	1095.154	1037.868	1018.159

TABLE B-3

RUN 2 PLAN 1

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.383	2.299	2.262	2.250	2.216	2.199
.670	2.266	2.175	2.141	2.133	2.112	2.037
.794	2.150	2.058	2.025	2.021	1.999	1.937
.918	2.050	1.958	1.925	1.925	1.908	1.891
1.041	1.950	1.858	1.833	1.833	1.821	1.803
1.165	1.850	1.758	1.733	1.737	1.725	1.716
1.289	1.750	1.658	1.633	1.642	1.633	1.625
1.412	1.675	1.591	1.566	1.571	1.562	1.562
1.536	1.608	1.525	1.504	1.508	1.500	1.500
1.660	1.558	1.483	1.458	1.466	1.458	1.458
1.783	1.525	1.450	1.433	1.433	1.429	1.425
1.907	1.466	1.404	1.383	1.383	1.383	1.379
2.153	1.408	1.350	1.333	1.337	1.333	1.329
2.401	1.358	1.304	1.291	1.291	1.287	1.287
2.650	1.308	1.250	1.241	1.241	1.237	1.237
2.899	1.241	1.195	1.183	1.191	1.183	1.183
3.393	1.125	1.091	1.079	1.083	1.079	1.079
3.888	1.033	1.016	1.004	1.008	1.008	1.008
4.383	.966	.950	.950	.950	.950	.950
4.878	.900	.891	.883	.883	.883	.883
4.980	.825	.825	.825	.825	.825	.825

TABLE B-3 (CONTINUED)

R FT.	PIEZOMETRIC SURFACE FT. AT 6 HR.	7 HR.	7.5 HR.
.397	2.792	2.792	2.792
.547	2.166	2.116	2.033
.670	2.062	2.016	1.937
.794	1.962	1.925	1.900
.918	1.875	1.842	1.821
1.041	1.796	1.766	1.745
1.165	1.703	1.683	1.666
1.289	1.616	1.591	1.579
1.412	1.550	1.533	1.521
1.536	1.491	1.475	1.453
1.660	1.450	1.433	1.425
1.783	1.421	1.403	1.399
1.907	1.375	1.366	1.358
2.153	1.325	1.316	1.303
2.401	1.283	1.275	1.266
2.650	1.233	1.225	1.216
2.899	1.183	1.175	1.166
3.393	1.079	1.075	1.066
3.888	1.003	.999	.999
4.383	.950	.941	.941
4.878	.883	.883	.883
4.930	.825	.825	.825

TABLE B-4

RUN 2 PLAN 1

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	777.041	614.302	569.531	568.785	567.351
1.412	896.242	746.030	704.232	706.373	714.142
2.341	1043.794	891.516	847.710	849.548	860.357
3.939	1193.739	1038.194	990.463	992.072	1006.134

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT 5 HR.	6 HR.	7 HR.	7.5 HR.
555.225	538.492	514.509	503.822
707.765	695.037	677.747	670.954
853.889	841.036	825.248	819.266
998.968	984.997	968.645	964.904

TABLE B-5

RUN 1 PLAN 2A

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.166	1.999	1.912	1.842	1.750	1.683
.670	2.091	1.925	1.833	1.775	1.675	1.603
.794	2.033	1.875	1.792	1.792	1.641	1.575
.918	1.983	1.825	1.741	1.683	1.599	1.537
1.041	1.916	1.775	1.699	1.641	1.562	1.500
1.165	1.867	1.725	1.658	1.604	1.525	1.466
1.289	1.817	1.683	1.616	1.566	1.495	1.433
1.412	1.766	1.641	1.583	1.537	1.475	1.416
1.536	1.691	1.575	1.520	1.479	1.416	1.365
1.660	1.633	1.525	1.475	1.437	1.379	1.329
1.783	1.583	1.483	1.433	1.399	1.345	1.299
1.907	1.525	1.433	1.391	1.358	1.316	1.266
2.153	1.450	1.366	1.325	1.299	1.254	1.216
2.401	1.399	1.316	1.291	1.262	1.225	1.187
2.650	1.333	1.258	1.233	1.212	1.175	1.141
2.899	1.266	1.199	1.133	1.162	1.133	1.103
3.393	1.141	1.099	1.083	1.070	1.050	1.029
3.888	1.058	1.025	1.012	1.004	.991	.974
4.383	.991	.966	.958	.954	.941	.933
4.878	.908	.900	.895	.891	.833	.879
4.930	.825	.825	.825	.825	.825	.825

TABLE B-5 (CONTINUED)

R FT.	PIEZOMETRIC SURFACE FT. AT 6 HR.	7 HR.	7.5 HR.
.397	2.792	2.792	2.792
.547	1.591	1.533	1.500
.670	1.525	1.475	1.441
.794	1.491	1.441	1.416
.918	1.458	1.408	1.383
1.041	1.425	1.383	1.354
1.165	1.391	1.354	1.325
1.289	1.366	1.333	1.304
1.412	1.358	1.316	1.291
1.536	1.304	1.266	1.250
1.660	1.275	1.233	1.221
1.783	1.245	1.208	1.195
1.907	1.221	1.191	1.171
2.153	1.166	1.141	1.125
2.401	1.150	1.116	1.108
2.650	1.108	1.083	1.075
2.899	1.075	1.050	1.041
3.393	1.008	.991	.983
3.888	.958	.950	.941
4.383	.916	.912	.908
4.878	.875	.871	.867
4.980	.825	.825	.825

TABLE B-6

RUN 1 PLAN 2A

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	1018.321	751.120	663.320	596.613	524.280
1.412	1357.948	1071.162	964.622	882.280	789.236
2.341	1537.101	1254.717	1161.548	1075.724	978.717
3.939	1739.068	1457.795	1361.189	1270.802	1162.943

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT 5 HR.	6 HR.	7 HR.	7.5 HR.
458.680	389.640	335.721	328.516
696.076	597.558	516.839	507.486
873.383	758.900	660.430	650.699
1051.913	922.416	809.538	800.293

TABLE B-7

RUN 2 PLAN 2A

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.266	2.108	2.075	2.058	2.025	1.983
.670	2.191	2.033	2.004	1.991	1.966	1.929
.794	2.125	1.966	1.945	1.937	1.916	1.879
.918	2.050	1.900	1.879	1.875	1.858	1.825
1.041	1.955	1.821	1.800	1.800	1.792	1.758
1.165	1.867	1.733	1.716	1.725	1.716	1.691
1.289	1.775	1.641	1.641	1.637	1.637	1.616
1.412	1.708	1.591	1.579	1.587	1.583	1.566
1.536	1.658	1.550	1.533	1.541	1.541	1.525
1.660	1.616	1.512	1.500	1.508	1.508	1.491
1.783	1.575	1.475	1.466	1.475	1.475	1.458
1.907	1.525	1.425	1.416	1.421	1.421	1.416
2.153	1.475	1.383	1.375	1.383	1.383	1.366
2.401	1.425	1.333	1.325	1.333	1.333	1.325
2.650	1.358	1.283	1.275	1.283	1.283	1.275
2.899	1.308	1.237	1.225	1.233	1.241	1.233
3.393	1.170	1.116	1.108	1.116	1.116	1.108
3.888	1.050	1.008	1.008	1.008	1.008	1.008
4.383	.958	.941	.933	.933	.933	.933
4.878	.900	.883	.883	.883	.883	.883
4.980	.825	.825	.825	.825	.825	.825

TABLE B-7 (CONTINUED)

	PIEZOMETRIC SURFACE FT. AT		
R FT.	6 HR.	7 HR.	7.5 HR.
.397	2.792	2.792	2.792
.547	1.958	1.916	1.900
.670	1.904	1.867	1.850
.794	1.858	1.817	1.808
.918	1.808	1.771	1.758
1.041	1.741	1.712	1.699
1.165	1.675	1.650	1.637
1.289	1.608	1.583	1.566
1.412	1.554	1.533	1.525
1.536	1.516	1.495	1.487
1.660	1.483	1.466	1.458
1.783	1.450	1.433	1.425
1.907	1.408	1.391	1.333
2.153	1.358	1.350	1.341
2.401	1.316	1.308	1.299
2.650	1.266	1.258	1.250
2.899	1.225	1.216	1.208
3.393	1.108	1.099	1.095
3.888	1.008	.999	.999
4.383	.933	.933	.929
4.878	.883	.883	.879
4.980	.825	.825	.825

TABLE B-3

RUN 2 PLAN 2A

	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	630.269	447.921	427.917	427.642	420.378
1.412	766.187	600.742	570.556	587.653	585.735
2.341	892.964	709.638	688.636	693.757	691.910
3.939	995.962	811.365	793.242	796.714	794.113

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT			
5 HR.	6 HR.	7 HR.	7.5 HR.
402.213	387.995	364.863	357.467
557.622	542.869	517.920	519.342
678.144	661.443	635.404	626.019
730.865	764.156	736.171	727.179

TABLE B-9

RUN 1 PLAN 3

R FT.	PIEZOMETRIC SURFACE IN FEET AT					
	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.193	2.145	2.116	2.095	2.087	2.075
.670	2.095	2.066	2.033	2.025	2.025	2.004
.794	2.033	1.999	1.973	1.966	1.966	1.954
.918	1.950	1.925	1.908	1.900	1.900	1.891
1.041	1.883	1.858	1.846	1.842	1.842	1.833
1.165	1.770	1.741	1.733	1.725	1.737	1.733
1.289	1.666	1.650	1.641	1.641	1.650	1.641
1.412	1.616	1.591	1.583	1.583	1.591	1.591
1.536	1.566	1.545	1.541	1.545	1.550	1.550
1.660	1.525	1.500	1.500	1.508	1.516	1.512
1.783	1.483	1.462	1.462	1.466	1.475	1.475
1.907	1.441	1.416	1.416	1.425	1.425	1.425
2.153	1.399	1.375	1.375	1.383	1.391	1.391
2.401	1.358	1.333	1.333	1.341	1.350	1.350
2.650	1.308	1.283	1.287	1.295	1.304	1.304
2.899	1.266	1.241	1.241	1.250	1.258	1.258
3.393	1.150	1.129	1.129	1.137	1.141	1.141
3.888	1.033	1.021	1.021	1.025	1.029	1.033
4.383	.958	.958	.954	.954	.954	.958
4.878	.900	.900	.895	.900	.900	.900
4.980	.825	.825	.825	.825	.825	.825

TABLE B-9 (CONTINUED)

	PIEZOMETRIC SURFACE FT. AT			
R FT.	6 HR.	7 HR.	8 HR.	9 HR.
.397	2.792	2.792	2.792	2.792
.547	2.041	2.016	1.983	1.958
.670	1.974	1.941	1.912	1.883
.794	1.925	1.900	1.867	1.842
.918	1.867	1.837	1.808	1.783
1.041	1.813	1.783	1.758	1.733
1.165	1.712	1.683	1.662	1.641
1.289	1.625	1.599	1.583	1.558
1.412	1.575	1.558	1.541	1.521
1.536	1.533	1.516	1.500	1.479
1.660	1.500	1.483	1.466	1.445
1.783	1.466	1.450	1.429	1.412
1.907	1.416	1.408	1.391	1.375
2.153	1.383	1.366	1.350	1.337
2.401	1.341	1.325	1.316	1.299
2.650	1.299	1.283	1.271	1.258
2.899	1.254	1.241	1.233	1.216
3.393	1.141	1.133	1.125	1.116
3.888	1.025	1.025	1.016	1.012
4.383	.958	.954	.950	.941
4.878	.900	.900	.891	.891
4.980	.825	.825	.825	.825

TABLE B-10

RUN 1 PLAN 3

	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	576.172	487.912	455.559	447.823	458.429
1.412	723.531	635.658	589.701	598.091	612.439
2.341	863.028	750.430	713.127	708.647	727.912
3.939	987.092	865.699	821.846	814.820	834.727

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
5 HR.	6 HR.	7 HR.	8 HR.	9 HR.
459.336	443.396	422.745	395.662	378.407
619.809	594.689	574.350	544.180	524.729
736.613	724.298	702.791	669.203	643.167
845.293	832.855	812.212	775.778	754.344

TABLE B-11

RUN 2 PLAN 3

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.308	2.262	2.221	2.183	2.145	2.104
.670	2.208	2.166	2.133	2.099	2.066	2.025
.794	2.133	2.099	2.075	2.050	2.016	1.983
.918	2.075	2.045	2.021	1.999	1.970	1.937
1.041	2.025	1.999	1.974	1.950	1.925	1.900
1.165	1.995	1.958	1.941	1.916	1.900	1.867
1.289	1.808	1.771	1.750	1.729	1.708	1.683
1.412	1.758	1.716	1.699	1.683	1.666	1.650
1.536	1.691	1.671	1.658	1.645	1.625	1.608
1.660	1.658	1.637	1.625	1.616	1.599	1.579
1.783	1.633	1.608	1.599	1.591	1.575	1.558
1.907	1.575	1.550	1.537	1.525	1.516	1.500
2.153	1.520	1.500	1.491	1.483	1.475	1.458
2.401	1.475	1.454	1.445	1.437	1.425	1.416
2.650	1.416	1.395	1.391	1.383	1.375	1.362
2.899	1.358	1.341	1.341	1.333	1.325	1.316
3.393	1.233	1.225	1.216	1.216	1.208	1.199
3.888	1.125	1.108	1.108	1.108	1.099	1.095
4.383	1.025	1.016	1.016	1.016	1.012	1.008
4.878	.954	.950	.945	.945	.945	.941
4.980	.825	.825	.825	.825	.825	.825

TABLE B-11 (CONTINUED)

R FT.	PIEZOMETRIC SURFACE FT. AT 6 HR.	7 HR.	8 HR.	9 HR.
.397	2.792	2.792	2.792	2.792
.547	2.058	2.012	1.978	1.941
.670	1.983	1.941	1.903	1.871
.794	1.933	1.891	1.858	1.817
.918	1.895	1.854	1.817	1.787
1.041	1.858	1.821	1.792	1.758
1.165	1.837	1.800	1.771	1.733
1.289	1.650	1.621	1.595	1.566
1.412	1.625	1.595	1.571	1.550
1.536	1.579	1.550	1.516	1.500
1.660	1.550	1.525	1.495	1.475
1.783	1.533	1.504	1.475	1.458
1.907	1.475	1.450	1.433	1.408
2.153	1.437	1.412	1.391	1.366
2.401	1.395	1.375	1.358	1.333
2.650	1.345	1.325	1.308	1.291
2.899	1.299	1.283	1.266	1.250
3.393	1.191	1.175	1.166	1.150
3.888	1.091	1.079	1.071	1.062
4.383	.999	.995	.991	.983
4.878	.937	.933	.933	.925
4.980	.825	.825	.825	.825

TABLE B-12

RUN 2 PLAN 3

	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	726.535	655.085	615.741	595.311	579.079
1.412	879.426	806.989	774.180	762.229	754.185
2.341	1012.202	938.365	906.237	897.009	894.113
3.939	1121.746	1047.997	1013.931	1007.743	1007.183

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
5 HR.	6 HR.	7 HR.	8 HR.	9 HR.
549.741	500.924	466.937	435.529	409.809
729.677	679.632	650.846	606.076	578.127
871.096	818.212	781.335	741.806	710.171
985.722	932.334	896.690	856.653	824.795

TABLE B-13

RUN 1 PLAN 2B

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	0.5 HR.	1 HR.	1.5 HR.	2 HR.	2.5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.266	2.208	2.183	2.141	2.108	2.071
.670	2.208	2.150	2.125	2.083	2.050	2.016
.794	2.141	2.083	2.058	2.025	1.991	1.962
.918	2.083	2.025	2.008	1.974	1.941	1.916
1.041	2.025	1.974	1.954	1.925	1.900	1.871
1.165	1.983	1.941	1.916	1.883	1.863	1.837
1.289	1.941	1.900	1.875	1.850	1.825	1.804
1.412	1.891	1.850	1.829	1.804	1.787	1.762
1.536	1.850	1.800	1.783	1.766	1.745	1.725
1.660	1.808	1.766	1.750	1.725	1.712	1.691
1.783	1.775	1.737	1.721	1.699	1.683	1.666
1.907	1.683	1.654	1.633	1.616	1.608	1.583
2.153	1.616	1.587	1.566	1.554	1.541	1.525
2.401	1.554	1.529	1.508	1.491	1.483	1.471
2.650	1.475	1.458	1.441	1.425	1.416	1.399
2.899	1.408	1.391	1.375	1.366	1.358	1.345
3.393	1.258	1.250	1.233	1.225	1.225	1.216
3.888	1.133	1.121	1.108	1.108	1.099	1.099
4.383	1.025	1.025	1.016	1.012	1.008	1.008
4.878	.950	.941	.941	.941	.941	.937
4.980	.825	.825	.825	.825	.825	.825

TABLE B-13 (CONTINUED)

R FT.	PIEZO. SUR. FT. AT 3 HR.	PIEZO. SUR. FT. AT 3.75 HR.
.397	2.792	2.792
.547	2.041	2.004
.670	1.983	1.950
.794	1.933	1.900
.918	1.887	1.858
1.041	1.842	1.817
1.165	1.813	1.783
1.289	1.775	1.750
1.412	1.737	1.712
1.536	1.699	1.679
1.660	1.666	1.650
1.783	1.645	1.625
1.907	1.566	1.550
2.153	1.508	1.491
2.401	1.458	1.437
2.650	1.391	1.375
2.899	1.333	1.321
3.393	1.208	1.195
3.888	1.091	1.083
4.383	1.004	.999
4.878	.933	.933
4.980	.825	.825

TABLE B-14

RUN 1 PLAN 2B

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	.5 HR.	1 HR.	1.5 HR.	2 HR.
.719	814.490	738.679	700.032	659.602	624.636
1.412	1086.320	1014.313	968.613	927.613	892.160
2.341	1213.618	1126.419	1081.264	1047.063	1015.991
3.939	1259.047	1214.577	1174.503	1144.714	1115.358

(CONTINUED)

PERM. COEF. (GAL./DAY SQ.FT.) AT 2.5 HR.	3 HR.	3.75 HR.
603.892	570.284	551.780
872.457	831.012	811.854
1000.686	963.784	947.975
1106.245	1070.417	1059.364

TABLE B-15

RUN 2 PLAN 2B

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	0.5 HR.	1 HR.	1.5 HR.	2 HR.	2.5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.266	2.125	2.083	2.041	2.012	1.983
.670	2.208	2.058	2.025	1.983	1.954	1.925
.794	2.141	1.991	1.958	1.921	1.900	1.867
.918	2.083	1.941	1.908	1.875	1.850	1.825
1.041	2.025	1.891	1.858	1.829	1.808	1.779
1.165	1.983	1.858	1.833	1.804	1.775	1.754
1.289	1.941	1.829	1.804	1.779	1.750	1.733
1.412	1.891	1.779	1.758	1.733	1.708	1.691
1.536	1.850	1.733	1.708	1.683	1.666	1.650
1.660	1.808	1.699	1.675	1.650	1.637	1.616
1.783	1.775	1.675	1.654	1.625	1.612	1.591
1.907	1.683	1.591	1.566	1.554	1.533	1.516
2.153	1.616	1.533	1.516	1.491	1.483	1.462
2.401	1.554	1.475	1.458	1.441	1.433	1.412
2.650	1.475	1.408	1.395	1.379	1.366	1.350
2.899	1.408	1.350	1.337	1.325	1.316	1.299
3.393	1.258	1.216	1.208	1.199	1.191	1.183
3.888	1.133	1.099	1.095	1.091	1.083	1.075
4.383	1.025	1.008	1.008	.999	.995	.991
4.878	.950	.941	.937	.933	.933	.929
4.980	.825	.825	.825	.825	.825	.825

TABLE B-15 (CONTINUED)

	PIEZO. SUR. FT. AT	
R FT.	3 HR.	3.75 HR.
.397	2.792	2.792
.547	1.958	1.916
.670	1.900	1.863
.794	1.842	1.808
.918	1.800	1.766
1.041	1.758	1.725
1.165	1.733	1.699
1.289	1.708	1.683
1.412	1.671	1.641
1.536	1.625	1.599
1.660	1.599	1.571
1.783	1.575	1.550
1.907	1.504	1.483
2.153	1.450	1.429
2.401	1.399	1.383
2.650	1.350	1.325
2.899	1.291	1.275
3.393	1.175	1.166
3.888	1.075	1.066
4.383	.991	.983
4.878	.929	.925
4.980	.825	.825

TABLE B-16

RUN 2 PLAN 2B

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	.5 HR.	1 HR.	1.5 HR.	2 HR.
.719	814.490	635.394	590.465	553.553	539.575
1.412	1086.320	903.202	851.075	806.920	791.455
2.341	1213.618	1029.453	978.587	937.302	926.393
3.939	1259.047	1134.440	1085.896	1046.522	1038.802

(CONTINUED)

PERM. COEF. (GAL./DAY SQ.FT.) AT 2.5 HR.	3 HR.	3.75 HR.
509.708	492.641	465.472
755.293	733.732	698.568
887.458	869.854	834.754
1001.828	985.283	952.036

TABLE B-17

RUN 3 PLAN 2B

PIEZOMETRIC SURFACE IN FEET AT

R FT.	S S	0.5 HR.	1 HR.	1.5 HR.	2 HR.	2.5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.266	2.125	2.066	2.021	1.970	1.933
.670	2.208	2.025	1.970	1.941	1.883	1.850
.794	2.141	1.933	1.891	1.858	1.817	1.783
.918	2.083	1.875	1.833	1.800	1.762	1.729
1.041	2.025	1.825	1.783	1.750	1.716	1.687
1.165	1.983	1.792	1.750	1.712	1.683	1.658
1.289	1.941	1.754	1.716	1.683	1.654	1.629
1.412	1.891	1.716	1.675	1.641	1.616	1.595
1.536	1.850	1.662	1.633	1.608	1.575	1.554
1.660	1.808	1.633	1.599	1.575	1.550	1.525
1.783	1.775	1.608	1.575	1.550	1.525	1.508
1.907	1.683	1.533	1.500	1.479	1.454	1.437
2.153	1.616	1.475	1.445	1.425	1.404	1.387
2.401	1.554	1.421	1.395	1.375	1.358	1.350
2.650	1.475	1.358	1.337	1.321	1.304	1.241
2.899	1.408	1.308	1.283	1.271	1.254	1.241
3.393	1.258	1.187	1.175	1.158	1.145	1.141
3.888	1.133	1.083	1.075	1.066	1.054	1.050
4.383	1.025	.999	.991	.983	.978	.975
4.878	.950	.941	.933	.925	.925	.925
4.980	.825	.825	.825	.825	.825	.825

TABLE B-17 (CONTINUED)

R FT.	PIEZO. SUR. FT. AT 3 HR.	3.75 HR.
.397	2.792	2.792
.547	1.900	1.858
.670	1.817	1.779
.794	1.750	1.716
.918	1.699	1.666
1.041	1.658	1.625
1.165	1.633	1.599
1.289	1.599	1.575
1.412	1.566	1.541
1.536	1.533	1.504
1.660	1.500	1.479
1.783	1.483	1.458
1.907	1.416	1.395
2.153	1.366	1.350
2.401	1.325	1.308
2.650	1.275	1.258
2.899	1.233	1.216
3.393	1.133	1.116
3.888	1.041	1.033
4.383	.970	.966
4.878	.925	.916
4.980	.825	.825

TABLE B-18

RUN 3 PLAN 2B

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	.5 HR.	1 HR.	1.5 HR.	2 HR.
.719	814.490	544.857	439.108	456.589	424.925
1.412	1086.320	776.315	706.846	665.127	634.164
2.341	1213.618	904.566	832.498	791.437	755.970
3.939	1259.047	1018.554	946.527	904.040	869.114

(CONTINUED)

PERM. COEF. (GAL./DAY SQ.FT.) AT 2.5 HR.	3 HR.	3.75 HR.
407.446	384.529	361.808
615.859	576.317	546.246
730.142	700.880	668.886
843.435	814.669	780.951

TABLE B-19

RUN 1 PLAN 2A

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.299	2.266	2.233	2.191	2.191	2.133
.670	2.225	2.191	2.158	2.116	2.116	2.066
.794	2.166	2.125	2.099	2.066	2.062	2.008
.918	2.116	2.083	2.058	2.025	2.016	1.970
1.041	1.925	1.887	1.853	1.825	1.829	1.733
1.165	1.875	1.833	1.808	1.775	1.779	1.737
1.289	1.691	1.641	1.616	1.587	1.583	1.550
1.412	1.675	1.666	1.599	1.579	1.558	1.529
1.536	1.608	1.566	1.554	1.533	1.525	1.491
1.660	1.583	1.554	1.533	1.516	1.508	1.475
1.783	1.566	1.533	1.516	1.500	1.491	1.462
1.907	1.508	1.483	1.466	1.450	1.441	1.416
2.153	1.471	1.445	1.433	1.416	1.408	1.379
2.401	1.425	1.399	1.391	1.375	1.366	1.341
2.650	1.371	1.350	1.337	1.325	1.316	1.295
2.899	1.325	1.304	1.291	1.283	1.275	1.254
3.393	1.191	1.175	1.166	1.158	1.154	1.141
3.888	1.075	1.062	1.058	1.054	1.050	1.041
4.383	.991	.983	.974	.974	.974	.966
4.878	.925	.925	.925	.921	.916	.912
4.980	.825	.825	.825	.825	.825	.825

TABLE B-19 (CONTINUED)

R FT.	PIEZOMETRIC SURFACE FT. AT		
	6 HR.	7 HR.	7.5 HR.
.397	2.792	2.792	2.792
.547	2.108	2.133	2.108
.670	2.033	2.066	2.041
.794	1.991	2.021	1.991
.918	1.950	1.979	1.950
1.041	1.758	1.792	1.771
1.165	1.708	1.741	1.721
1.289	1.525	1.550	1.533
1.412	1.508	1.525	1.508
1.536	1.475	1.500	1.483
1.660	1.458	1.483	1.466
1.783	1.441	1.466	1.458
1.907	1.399	1.416	1.408
2.153	1.366	1.383	1.375
2.401	1.333	1.341	1.333
2.650	1.283	1.299	1.291
2.899	1.241	1.258	1.250
3.393	1.133	1.141	1.137
3.888	1.033	1.041	1.033
4.383	.958	.966	.966
4.878	.908	.916	.908
4.980	.825	.825	.825

TABLE B-20

RUN 1 PLAN 2A

	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	631.728	583.697	538.636	508.304	504.251
1.412	732.665	691.466	652.563	638.175	618.041
2.341	873.907	833.024	788.236	766.359	757.611
3.939	980.068	942.201	895.270	874.611	867.206

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT			
5 HR.	6 HR.	7 HR.	7.5 HR.
450.447	320.654	465.492	493.295
568.968	522.813	535.039	560.524
702.894	685.312	722.028	695.063
810.994	789.328	832.352	802.318

TABLE B-21

RUN 2 PLAN 2A

R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
	PIEZOMETRIC SURFACE IN FEET AT					
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.299	2.179	2.108	2.050	2.017	2.021
.670	2.225	2.091	2.025	1.962	1.933	1.941
.794	2.166	2.033	1.970	1.912	1.933	1.887
.918	2.116	1.987	1.925	1.367	1.883	1.846
1.041	1.925	1.792	1.741	1.691	1.716	1.679
1.165	1.875	1.741	1.691	1.641	1.658	1.629
1.289	1.691	1.550	1.508	1.475	1.483	1.453
1.412	1.675	1.525	1.479	1.445	1.458	1.433
1.536	1.608	1.491	1.454	1.421	1.433	1.421
1.660	1.583	1.475	1.441	1.408	1.416	1.412
1.783	1.566	1.458	1.425	1.391	1.404	1.337
1.907	1.508	1.408	1.375	1.350	1.358	1.350
2.153	1.471	1.375	1.341	1.316	1.325	1.316
2.401	1.425	1.341	1.308	1.387	1.291	1.233
2.650	1.371	1.291	1.266	1.241	1.250	1.241
2.899	1.325	1.250	1.225	1.208	1.216	1.204
3.393	1.191	1.137	1.116	1.108	1.108	1.104
3.888	1.075	1.037	1.025	1.016	1.016	1.003
4.383	.991	.966	.958	.950	.950	.950
4.878	.925	.916	.908	.904	.908	.903
4.980	.825	.825	.825	.825	.825	.825

TABLE B-21 (CONTINUED)

R FT.	PIEZOMETRIC SURFACE FT. AT 6 HR.	7 HR.	7.5 HR.
.397	2.792	2.792	2.792
.547	1.966	2.075	2.050
.670	1.891	1.979	1.953
.794	1.842	1.916	1.908
.918	1.804	1.875	1.858
1.041	1.641	1.708	1.683
1.165	1.595	1.650	1.633
1.289	1.433	1.483	1.466
1.412	1.412	1.458	1.441
1.536	1.391	1.433	1.425
1.660	1.375	1.416	1.408
1.783	1.366	1.399	1.391
1.907	1.325	1.358	1.350
2.153	1.299	1.325	1.316
2.401	1.266	1.291	1.283
2.650	1.225	1.254	1.241
2.899	1.191	1.216	1.208
3.393	1.087	1.112	1.108
3.888	1.008	1.021	1.008
4.383	.941	.954	.950
4.878	.900	.908	.900
4.980	.825	.825	.825

TABLE B-22

RUN 2 PLAN 2A

R FT.	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	631.728	434.913	383.320	351.486	379.204
1.412	732.665	539.484	497.922	463.421	494.622
2.341	873.907	665.788	619.686	586.160	619.306
3.939	980.068	769.344	722.248	682.800	724.895

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT 5 HR.	6 HR.	7 HR.	7.5 HR.
349.085	320.826	373.885	353.485
465.885	436.046	490.414	467.951
585.909	552.051	614.804	587.835
687.975	650.892	719.911	689.646

TABLE B-23

RUN 3 PLAN 2n

	PIEZOMETRIC SURFACE IN FEET AT					
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.	5 HR.
.397	2.792	2.792	2.792	2.792	2.792	2.792
.547	2.299	1.941	1.891	1.846	1.833	1.737
.670	2.225	1.850	1.796	1.750	1.733	1.699
.794	2.166	1.792	1.754	1.699	1.691	1.654
.918	2.116	1.758	1.716	1.666	1.658	1.621
1.041	1.925	1.591	1.550	1.516	1.516	1.483
1.165	1.875	1.550	1.508	1.475	1.475	1.441
1.289	1.691	1.391	1.358	1.333	1.325	1.308
1.412	1.675	1.375	1.341	1.316	1.312	1.291
1.536	1.608	1.358	1.329	1.299	1.295	1.275
1.660	1.583	1.341	1.316	1.291	1.283	1.266
1.783	1.566	1.333	1.308	1.283	1.275	1.258
1.907	1.508	1.291	1.266	1.241	1.241	1.225
2.153	1.471	1.266	1.245	1.225	1.216	1.199
2.401	1.425	1.237	1.216	1.199	1.191	1.175
2.650	1.371	1.199	1.183	1.166	1.158	1.150
2.899	1.325	1.166	1.150	1.133	1.133	1.121
3.393	1.191	1.075	1.066	1.050	1.050	1.041
3.888	1.075	.995	.991	.983	.983	.974
4.383	.991	.937	.933	.925	.925	.925
4.878	.925	.900	.891	.891	.891	.891
4.980	.825	.825	.825	.825	.825	.825

TABLE B-23 (CONTINUED)

	PIEZOMETRIC SURFACE FT. AT			
R FT.	6 HR.	7 HR.	7.5 HR.	F S
.397	2.792	2.792	2.792	2.792
.547	1.745	1.891	1.858	2.191
.670	1.658	1.771	1.741	2.075
.794	1.616	1.721	1.691	2.016
.918	1.587	1.675	1.650	1.966
1.041	1.458	1.529	1.508	1.792
1.165	1.416	1.487	1.462	1.741
1.289	1.283	1.341	1.316	1.541
1.412	1.266	1.325	1.299	1.516
1.536	1.258	1.308	1.291	1.491
1.660	1.250	1.291	1.275	1.475
1.783	1.237	1.283	1.266	1.458
1.907	1.208	1.250	1.233	1.416
2.153	1.183	1.225	1.208	1.375
2.401	1.162	1.199	1.183	1.333
2.650	1.133	1.166	1.150	1.291
2.899	1.108	1.137	1.125	1.241
3.393	1.033	1.058	1.041	1.133
3.888	.966	.983	.974	1.033
4.383	.916	.925	.925	.966
4.878	.883	.891	.887	.908
4.980	.825	.825	.825	.825

TABLE B-24

RUN 3 PLAN 2A

	PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
R FT.	S S	1 HR.	2 HR.	3 HR.	4 HR.
.719	631.728	300.715	274.927	246.919	251.761
1.412	732.665	414.351	384.749	350.676	357.966
2.341	873.907	528.047	492.595	450.971	460.632
3.939	980.068	628.313	589.035	541.757	554.373

(CONTINUED)

PERMEABILITY COEFFICIENT (GAL./DAY SQ.FT.) AT				
5 HR.	6 HR.	7 HR.	7.5 HR.	F S
230.927	216.463	264.157	243.554	439.763
332.160	314.096	372.424	345.630	549.865
428.866	406.708	478.387	445.145	668.273
517.954	492.559	574.394	536.508	784.206